ENGINEERING PAPERS

BY

GRAHAM SMITH.

Engineering

F. W. Maher
Dec 9/83.

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MORTAR:

A Paper read at a Supplemental Meeting of the Institution of Civil Engineers, May 23rd, 1873,—to which the Council awarded a "Miller Prize,"—afterwards read with the Addenda at the Edinburgh and Leith Engineers' Society.

PRACTICAL IRONWORK:

A Paper read at a Supplemental Meeting of the Institution of Civil Engineers, April 24th, 1874,—to which the Council awarded a "Miller Prize."

RETAINING WALLS:

A Paper read at a Meeting of the Edinburgh and Leith Engineers' Society, May 6th, 1874.

WITH ADDENDA.

BY

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TO

JOHN FOWLER, Esq., F.G.S.,

PAST PRESIDENT OF THE INSTITUTION OF CIVIL ENGINEERS,

&c. &c. &c.

THESE PAPERS

ARE

Dedicated,

AS A TRIBUTE OF ESTEEM,

AND A SLIGHT EXPRESSION OF GRATITUDE

FOR HIS MANY ACTS OF KINDNESS

TO

THE AUTHOR.

PREFACE.

In submitting these papers to the public, it has been deemed advisable strictly to adhere to the originals; therefore, with the exception of the Addenda to Mortar, they are substantially as read. The allotted time for reading at the meetings being limited they are necessarily short, yet by omitting all superfluities a considerable amount of matter has been condensed into them.

In connection with all subjects there are many points which it is impossible to touch upon when reading a paper. Statements involving numerical results, which may be of great value, are very uninteresting and generally entirely fail to command the attention of an audience; some of these closely bearing on the subject-matter have been introduced at the end of each paper, along with such other explanations as have seemed necessary, and it is trusted they will render the whole of interest to engineering students.

The opinions elicited in the discussions are appended, with the author's replies fully worked out by subsequent thought and experience.

Should this volume meet with the same reception from the public as the papers have already received from the scientific press, and from those to whom they have been addressed, the author will feel that his slight endeavours to promote engineering science have been amply rewarded.

Dock Yard, Liverpool,
January 1, 1875.

SYNOPSIS OF CONTENTS.

MORTAR.

ADDENDA.

Discussion.

PRACTICAL IRONWORK.

Addenda.

Discussion.

RETAINING WALLS.

Addenda.

Discussion.

ENGINEERING PAPERS.

MORTAR.

Being a paper read at a Supplemental Meeting of the Institution of Civil Engineers, May 23rd, 1873; George Barclay Bruce, Esq., *Member of Council, in the Chair.*

In buildings and structures, mortar is employed as the agent for causing the stones, bricks, and other materials used in construction, to adhere together, also to fill any crevices and irregularities in bedding them. Its use for these purposes is of the remotest antiquity; we read of slime being used in building the Tower of Babel, and asphalte in the construction of the walls of Babylon; and it is found, from an analysis of mortar taken from the pyramids of Cheops, that the Egyptians employed lime and sand in almost exactly the same proportions that we now do; and even the careful directions given by Vitruvius in the fifteenth century were carried out until the more modern researches of Vicat.

The remarks in this paper will be confined to the treatment of mortar formed by the admixture of lime with sand, and other ingredients; and, as it is the author's impression that a few facts obtained from actual practice are of much more value than any number of individual opinions which he might offer, he will, by the kind permission of George Fosbery Lyster, Esq., member of this Institution, and Engineer-in-

Chief to the Mersey Docks and Harbour Board, endeavour to base this paper on data obtained whilst studying the method carried out by that gentleman at Liverpool.

The limestone, which has been here employed for the past forty years in carrying out the most extensive hydraulic works, is obtained from quarries situate in the Halkin Mountains, Flintshire, and is that ordinarily used in Lancashire, Cheshire, the West of England, and North Wales.

It is found, from an analysis by Dr. Musprat, of Liverpool, to be composed of 75 per cent. of substances soluble in nitric and hydrochloric acids,* and 25 per cent. of those insoluble. The soluble substances are:

	Per Cent.
Carbonate of lime	72·0
Carbonate of magnesia	1·3
Proto-carbonate of iron .. .. } Sulphide of iron .. .. }	1·0
Alkalies	0·7

Those insoluble:

Silicic acid	20·0
Alumina	3·5
Sesquioxide of iron	1·1
Water and carbonaceous matter..	0·4

The limestone, in order to expel carbonic acid, is calcined in open kilns, on plan oval 16′ 6″ × 13′ 0″, and 26′ 0″ in height from the firebars, which dimensions allowing for a slight taper to the sides give a capacity of 3400 cubic feet; the interior is lined with fire bricks. Three such kilns are built into one rectangular construction of rubble work, each of which is provided with a hoist for the purpose of lifting the limestone and fuel to the summit of the structure when filling the kiln. The charging is done in the following manner: a few shavings are placed uponthe fire bars, upon

* Hydrochloric acid in its crude or impure state, is known as muriatic acid.

which is spread a layer of coke about 6 inches in thickness, limestone is then thrown in until a thickness of 1′ 10″ or 2′ 0″ is attained, this is followed by another layer of coke, and so on, alternate layers of coke and stone, until the top of the kiln is reached, the layers of stone gradually increasing in thickness to 2′ 6″ at the top; with the exception of the uppermost, which owing to its being exposed to the atmosphere, is made only 9 or 12 inches; when completely charged the shavings are lighted, and the whole allowed to burn for six or seven days, as experience may direct, after which time little trace of the coke is perceptible; the fire-bars are then withdrawn, and the burnt lime raked out of the aperture thus formed on to the floor of an adjoining shed, where it is slaked with water, and produces a lime of moderate whiteness; after which, owing to the irregular size of the stones put into the kiln, it is occasionally found that some of them are not sufficiently burnt; when this happens they are picked out and reburnt; but by care in having the stones reduced to about the same size in the first instance this seldom occurs.

In burning lime care must always be taken not to reach too high a temperature, as, owing to the fluxing properties of the lime, the silica and alumina would combine and form a species of glass; the stones should also be broken to a comparatively small size, in order that the heat may more readily penetrate to their interior and thus effect a saving in fuel.

The amount of quarried limestone put into the kiln is 110 tons, and the requisite amount of coke is 12½ to 14½ tons; this produces 75 tons, or 2170 bushels of burnt lime, which, with 31 tons of water necessary to slake this quantity, yields 98 tons of slaked lime, or 3411 bushels. From these quantities it will be seen that the slaked lime has a little over 1½ times the volume of the burnt lump lime which produced it.

The prices paid for the same lime differ very considerably in various localities; but it may be well to state that the cost of Halkin lime in Liverpool, in 1873, was 17*s.* 6*d.* per yard, or 10*d.* per bushel.

Limestones, when calcined, produce rich limes, hydraulic limes, and cements.

Rich limes are produced from stones consisting almost wholly of carbonate of lime, such as chalk; they slake freely, and during this process augment from two and a half to three and a half times in volume; these harden slowly in air and not at all in water, and the mortar formed from these is liable to be affected by changes in the atmosphere.

Hydraulic lime is obtained from stones containing 15 to 30 per cent. of silicates, and sometimes magnesia; these do not slake freely, give off little heat, and will harden slowly under water.

Some stones, containing 40 to 60 per cent. of silicates, produce cements which do not slake, but which, when ground and mixed with water, will set in air or water in a few minutes.

The agency to which mortars owe their power of setting is not generally understood, but it is commonly considered that this action in rich limes is due to the evaporation of water, and the gradual absorption of carbonic acid from the atmosphere, thus forming a crystallized carbonate of lime. In hydraulic limes it is believed that the setting takes place from a chemical union of the lime with silica and alumina, thus forming an insoluble crystallized double silicate, without which mortars placed in positions where air cannot penetrate would never harden.

The author, therefore, considers that the quantity of carbonic acid gas in the atmosphere being limited will, to some extent, account for the slow setting of rich limes; and, as the atmosphere cannot penetrate to a great extent into thick

walls and masses of concrete, it would be unadvisable to use for these purposes anything but hydraulic limes or cements, for the hardening of which the influence of the atmosphere is comparatively unimportant.

From the analysis of the Halkin limestone, it will be seen that the components producing setting and indurating under water exist to the extent of 25 per cent.; and being evenly distributed through its entire mass, produce a mortar most favourable to the formation of an insoluble crystallized double silicate.

To obtain good mortar, as much depends on the character of the ingredients and the manner of mixing them as on the goodness of the lime itself; it does not necessarily follow that because a lime is good that the quality of the mortar will be good also; the best lime ever burnt would be spoilt by the custom, common among some builders, of mixing with it alluvial soil and rubbish taken from the foundation pits of intended buildings. The sand should be hard, sharp, gritty, and, for engineering purposes, not too fine; it should be perfectly free from all organic matter, and with no particular smell; good sand for mortar may be rubbed between the hands without soiling them.

The water should also be free from all organic matter, and on this account should never be taken from stagnant ponds.

The presence of salt in sand and water is not found to impair the ultimate strength of most mortars, nevertheless it causes the work to *nitrate*, or, as it is commonly termed, *saltpetre*, which consists of white frothy blotches appearing on the face of the structure; it also renders the mortar liable to moisture, and for these reasons should never be present in mortar intended for architectural purposes, although for dock walls and sea works it may generally be used with advantage and economy.

Sand is used to increase the resistance of mortar to crush-

ing, to lessen the amount of shrinking, and to reduce the bulk of the more costly material, lime. Water is the agent by which a combination is effected, and as sand does not increase in volume by moisture, it necessarily follows that no more of the aqueous element should be employed than is absolutely necessary to fill the interstices between the sand, and render the whole into a paste convenient for use; and the greater strictness with which this is adhered to the more compact and durable will be the mortar.

The mortar made from the Halkin lime is mostly employed on the Mersey Dock Estate, in the construction of dock and river walls, for which purposes it is always mixed with salt water and sea-sand. The lime, within one to four days after being slaked, is taken to the mortar mills, which are cast-iron circular pans, 7′ 0″ in diameter, caused to revolve by suitable spur gearing at the rate of twenty revolutions a minute; in each pan are placed two rolling stones 4′ 6″ in diameter. There are fourteen such mills to each set of three kilns, which are driven by an engine of 50 indicated H P, and it is generally calculated that one mill requires $3\frac{1}{2}$ to 4 HP to work it; as the mills are seldom all working at the same time, the engine before mentioned is found adequate to drive the mills, lift the stone and fuel to the top of the kilns, and to pump from an adjacent dock the required quantity of water for mixing the mortar. The pans of the mills are provided with false bottoms, in order that they may be replaced when worn out, the average life of these being about three months.

In mixing the mortar the lime is first ground in the mills in a dry state for three minutes, the sand is then added, and after five minutes from the commencement the water is turned on; and as the necessary quantity is gauged by the tap it is allowed to run the whole time, which, for the ordinary mortar, is about thirty minutes, the quantity made at each mill in this time is a quarter of a cubic yard; in some cases the amount

is actually measured in order to ascertain if the men are making their full quantity; one man has to carry from an adjoining shed sufficient lime, sand, and ashes to make 5 cubic yards of mortar in a day, for which he is paid 3*s.* 6*d.*

The ordinary mortar used in the construction of rubble masonry for dock walls is mixed by volume in the following proportions: 1 part slaked lime, 2 parts sand, and $\frac{1}{3}$ of a part smithy ashes.

And the proportions for that used in brickwork are—1 slaked lime, 1 sand, and 1 smithy ashes.

For the sake of convenience in laying before you the experimental results obtained with these compositions, the author will term them respectively *masons' mortar* and *bricklayers' mortar.*

In practice the ingredients are not measured, as it is found that three average spades of lime, sand, or ashes, are equivalent to one bushel.

The mode of testing pursued was as follows: bricks, the quality of which will be described in each individual case, were accurately cut to $4\frac{1}{4}$ inches in width; these were in all cases thoroughly wetted and bedded crossways with a mortar joint $\frac{5}{16}''$ thick and $4\frac{1}{4}'' \times 4\frac{1}{4}''$, giving a testing area of 18 square inches. On the time arriving for testing, which, unless otherwise mentioned, was in every instance 168 days, or six lunar months, stirrups were passed round the ends of the bricks; two of these were attached to a beam, and on the remaining two a bucket was hung, into which perfectly dry sand was allowed to run from a hopper, the door of which was immediately closed when the joint parted, the bucket and sand were then weighed, and this was taken to be the breaking weight of the specimen.

In order to ascertain the difference which would exist in practice from the employment of bricks of various textures, two qualities were experimented upon, namely, common

bricks, similar to, although slightly harder than, those known about London as ordinary stocks; and fire bricks, very hard, and much the same as Staffordshire blue bricks.

The masons' mortar with common bricks broke with 496 lbs., with fire bricks, 433 lbs.; the bricklayers' mortar, with common bricks, 610 lbs.; with fire bricks, 516 lbs. These are the average results of three experiments in each instance, from which it would appear that soft porous bricks are preferable for work subjected in any way to a tensile strain.

It being the author's impression that mortar when used in a structure would bear a greater test, owing to the compression caused by the weight of the superincumbent mass, some results were obtained by subjecting the samples, twenty-four hours after being bedded, to a pressure of 56 lbs., and following this up with an additional 56 lbs. every day, until 4 cwts. were placed upon each; the masons' mortar, under these conditions, with common bricks, broke with 683 lbs.; with fire bricks, 403 lbs; the bricklayers' mortar, with common bricks, 372 lbs.; with fire bricks, 423 lbs. These are not average results, one experiment only having been made with each. The first instance is the only case in which the author's theory holds good, the remaining three cases being considerably below the respective averages of 433, 610, 516 lbs. before mentioned. This may be accounted for, as the author fears that in placing on the weights the mortar was disturbed after having partially set, in which case it will never bind together a second time.

In the case of mortar remixed with water six days after the first mixing, it was found that with common bricks the masons' mortar broke with 432 lbs., against 496 lbs. obtained with the same mortar when first mixed; the bricklayers' mortar broke with 440 lbs., against 610 lbs. The advantage is thus shown of using mortar when first mixed.

The importance of the admixture of ashes with mortar to

be atmospherically dried, will be shown by the following results: The bricklayers' mortar with common bricks, after a lapse of eighty-four days, broke with 570 lbs.; when sand was substituted in place of ashes, that is, when the proportions were 1 slaked lime, 2 sand, and no ashes, it only required 257 lbs. to tear asunder the bricks. These are the averages of three experiments. This is no doubt attributable to the ashes being porous; they thus allow greater facilities for the absorption of carbonic acid from the atmosphere.

By testing with a Michell's Lever Cement Testing Machine, one of which is now before you, brickettes having a testing section of $1\frac{1}{2}'' \times 1\frac{1}{2}'' = 2 \cdot 25$ square inches, the average result of three experiments was found to be 248 lbs., which will compare very favourably with the results obtained by Mr. Grant with Portland cement, mixed in the proportion 3 of sand to 1 of cement, which broke with an average of 270 lbs. From the foregoing it will be seen that nothing like these high results can be depended upon in actual practice, as the maximum breaking weight with bricks and mortar was 780 lbs., or 43·3 lbs. to the square inch, against 110 lbs. obtained by breaking brickettes.

Although no experimental tests have been made with this mortar of any great age, still, from the pulling down of old work, it may with confidence be asserted that it fully complies with the old Scotch rhyme—

> "When a hundred years are past and gane,
> Then good mortar grows into stane."

On the Mersey Dock Estate every stone and brick is properly bedded, jointed, and covered with mortar and *grout*, which is simply the mortar reduced by water to a creamy consistency; it is poured over the work, and penetrates into the body of the masonry, thus filling all cavities and assisting to keep the work moist during its progress, thereby producing

an even settlement. It may be well to mention that this work is not done by contract, in which case the author considers so free a use of "grout" would not be advisable, as probably it would be made to perform imperfectly what ought to be done thoroughly with mortar. When using bricks they are in all cases moistened, as, if set dry or warm, the mortar would be robbed by absorption of the necessary moisture for its proper hardening.

From practice it is found that a cubic yard of rubble work contains one-third, and brickwork one-fourth of a cubic yard of mortar.

Note.—This paper was concluded by a few remarks on the Selenitic method of mixing mortar, the Practical Manager of the Company attending to give verbal explanations and illustrations; as without these they will be of little value, they are here omitted.

2, Queen's Square Place, Westminster,
March 26*th*, 1873.

Addenda.

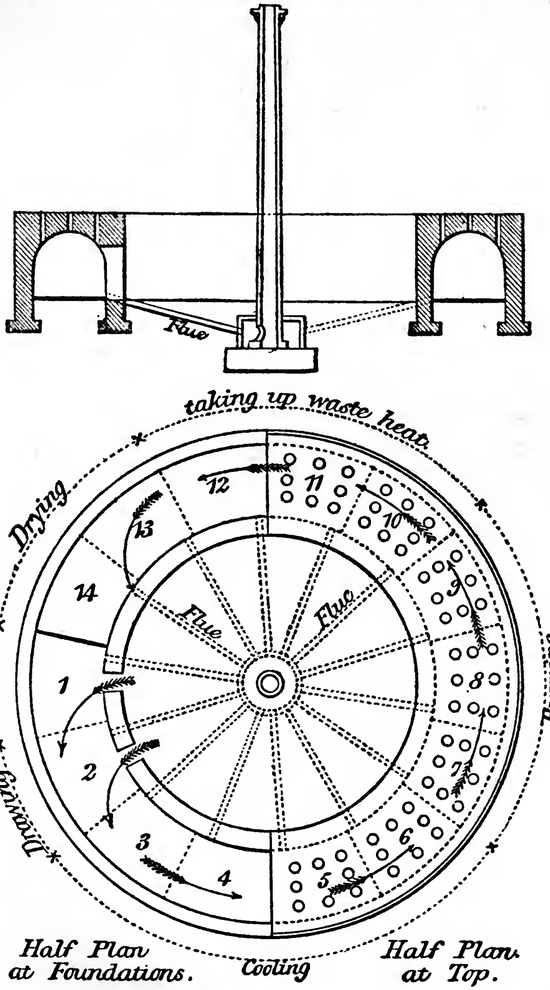
Read, March 18*th*, 1874, *along with the Original Paper, at a Meeting of the Edinburgh and Leith Engineers' Society;* Professor Fleeming Jenkin, *President, in the Chair.*

There are one or two other points which, with the permission of the meeting, the author will proceed to briefly mention.

The following description of Hoffman's Annular Kiln is taken nearly word for word from the 'Engineer' of Dec. 3rd, 1869. The slight alterations are necessary, as the author

considers the circular structure here shown to receive more favour than the oblong kiln there described.

SKETCH SHOWING PRINCIPLE OF HOFFMAN'S ANNULAR KILN.



The extraordinary and widespread success of these kilns, the beauty of the scientific principles they so ingeniously

embody, and the many fresh applications of which they are capable, are well deserving a careful study. The illustration shows half plan at foundation and half plan at top, which is sometimes at the level of the surrounding ground; the plan at top and also the section taken through the centre, show the holes down which the fuel is fed. The kiln substantially consists of a railway-tunnel-shaped passage, forming a long annular channel or ring. This ring is divided into twelve, or, as in the case of the kiln illustrated, fourteen compartments, which may be made to communicate or to be separated from each other by the raising or lowering of a partition or damper. The intercepting dampers are lowered in grooves built into the walls of the furnace immediately after each flue; or inserted from the side through the doorways. To each compartment there is an entrance doorway, which can be closed with temporary brickwork. Flues lead from the bed of each compartment to the central smoke chamber, which communicates with the chimney. The diagram indicates the action of a kiln constructed with fourteen compartments. In compartment No. 1 the green bricks are being set or stacked; from No. 2 they are being drawn, the air from the outside, entering here, cools the bricks in the compartments numbered 3, 4, 5, 6; in the compartments numbered 7, 8, 9, the bricks are being burnt, and it is into them that the fuel is being fed (the remaining chambers are entirely without fuel). This is done by simply dropping small pieces of fuel from the holes in the top amongst the already incandescent bricks, limestone, or other materials, which are stacked up in a simple way rendering this possible. The revolving draught, raised to a high temperature by previously passing through the incandescent bricks, here supplies the necessary oxygen to the fuel dropped in amongst them from above. The fire gases evolved from this fuel then pass into the compartments Nos. 10, 11, and 12, making the bricks therein ready for

burning. They at last reach the quite green bricks, out of which they absorb the moisture, driving it directly through the open valve of No. 14 into the central chimney. The state and progress of the fire can be at any time easily seen though the apertures on the top; and as the draught is under perfect control, the heat can be at once raised or lowered as may be required. The low temperature at which the gases leave the kiln is indicated by the fact that a high chimney for getting up the draught is required. There is thus a perpetual current, so to say, of bricks, which is brought slowly to revolve against, and in the contrary direction to, a perpetually revolving draught. In the green stages of the bricks they thus come in contact with air at a comparatively low temperature, and then gradually advance towards higher temperatures, until they are at last burnt. Each stack of bricks to be burnt is therefore most ingeniously made to combine successively—(1) the functions of a grate with bars; (2) of a Siemens' regenerator for heating the air to be burnt; (3) of a drying room for themselves when green; and (4) of a cooling room for themselves when burnt. It is impossible not to enjoy the examination of such an exemplification of the dominion of mind over matter.

Each day the fire is advanced, one chamber is emptied of its burnt and cooled down bricks, and another chamber is filled with green goods. The height of each chamber is only from 8 to 9 feet, or about that which a man can reach with his hands, so that the bricks or other materials can be easily taken in and out, and they are not subject to too great a weight when at a high temperature. Any repairs can also be at once done to any one of the chambers as it gets emptied in its turn.

In every process of treating clay, with a view to drying it in the open air or to burning it, it is absolutely necessary to carry out these operations gradually, to avoid cracking

and splitting the goods. This kiln, from its very gradual action, and absence of sudden changes of temperature, produces no *wasters* from these causes, and as the heat is very gradually brought to bear upon them, the bricks can also be burnt in a much more moist state than in ordinary kilns; though of course it is always best to set them when in as dry a state as possible.

By means of the existing air-tight envelope formed by the sand on the top and sides, and by hermetically closing the flues and feed pipes, the kilns can be closed up and left to themselves for five days and more. They can therefore be easily left alone for twenty-four hours, and thus they need not be fired on Sunday, which is frequently a very great convenience. They do not require skilled, or "Union" labour, and here science has again done away with that empirical skill, often acquired, and sometimes exerted, at the cost of much waste. The cost of labour is, in truth, reduced both as to quality and quantity.

In pottery and similar kilns the fuel is incased in open retorts, out of which the gases can flow without the fuel coming into contact with the goods.

During a visit to the Continent the author was invited to inspect one of these kilns near Boulogne, employed to burn bricks, and was supplied by the proprietor of the brickfield with the following particulars. The burning of 15,000 bricks requires three tons of coal, costing 90 francs, and the labour of two men at 10 francs, amounting in all to 100 francs, or 5*s.* 6*d.* per thousand, which he considered was about half the ordinary cost; whether the exact saving will reach this amount or not, is a question to be answered by those familiar with the working details of brick manufacture. But the economy of the system is almost beyond question; for, when visiting some large limeworks in the neighbourhood of Dorking, where two of these kilns were employed, the opinions of the continental brickmaker were fully endorsed

by the manager. In neither instance was any difficulty experienced in firing the kilns from the top with small fuel.

The lime burnt was that commonly known as "Dorking;" it is very slightly hydraulic, and is obtained by calcining the lower or grey chalks; it sometimes goes by the name of greystone in London, where, being cheap, it is largely in demand for ordinary building purposes, costing in the lump only 13*s.*, and when ground 15*s.* per yard.

More convincing evidence is given in the 'Practical Mechanics' Journal,' which, when speaking of these kilns, says:

"We have also been kindly supplied by Mr. Finlay, the manager of Mr. Bett's works, with a comparative statement of the cost of burning their bricks by the old kilns and by Hoffman's patent.

	Old Kilns.			Patent Kilns.		
	s.	*d.*		*s.*	*d.*	
Labour in burning	1	6	.. ..	0	6	per 1000
Coals, 10 cwts. 2 qrs.	7	10½	2 cwts.	1	6	"
Loss by waste, 10 per cent. ..	1	0	.. ..	0	0	"
	10	4½		2	0	

Saved by Hoffman's Patent 8*s.* 4½*d.* per 1000."

Although the first outlay on these kilns is great, being at least 50*l.* for each thousand bricks they are designed to burn per day, still, the before-mentioned results and opinions, together with the fact that the contractors for the Metropolitan and District Railways deemed it economical to set up kilns on this principle, solely to burn bricks for the construction of those railways, and removed them after their completion, will in the author's opinion fully sanction the large outlay where there is any amount of work to be got through.

The author, in so prominently placing these kilns before his readers, has no other motive than the desire to show his

appreciation of a valuable invention, and a wish to advance the interests of an important branch of the profession. Those wishing to form opinions for themselves will have every facility afforded them for so doing by referring to Mr. Hermann Wedekind, 3, Great Tower Street, London, E.C.

The fall of the Northfleet chimney, and the many letters written to the Editor of the 'Engineer' on the question, are no doubt fresh in the memory of most of the members present, but it may be well to reproduce a portion of a letter by one who states that he has had some "bitter experience in the erection of a shaft." ('Engineer,' Nov. 14th, 1873.)

"If the precaution was taken, in building these high chimneys, to have a fire placed at the bottom of the cavity, or in the shaft every night after the bricklayers had done their day's work, setting each night the work done the day previous, we should run less risks, and save the repetition of such dreadful calamities as the fall of the chimney at Northfleet."

The author being of opinion that this advice was most misleading, and would, if carried out, be the cause of augmenting such dreadful calamities, thought it well to reply; and although the text of his letter is embodied in this paper, he will quote a few lines, as the statements may be of more value, having remained uncontradicted by the criticizing public.

"The sun or heat will not set mortar, it simply dries it, and by so doing robs it of the necessary amount of moisture required for its proper hardening, and thus materially reduces its resistance to crushing as well as adhesive and cohesive strength. Mortar set in summer is not generally so good as that set in winter, of course leaving frost out of the question. The expedient of placing a fire in the shaft of a chimney, with the object of 'setting each night the work done the previous day,' should never be resorted to. Mortar should never be hardened by heat; it would be equally justifiable to use warm bricks, which, it is needless to state, is contrary to sound engineering practice and the theory of mortar."

We are told the design of this chimney was considerably more stable than many similar ones now standing; that it

was constructed on a solid bed of chalk, with the best bricks, in the most approved manner; that the work was not pushed, and that there was not the slightest scamping, all of which statements evidence proves, there is little reason to doubt; and to make things doubly sure, the mortar used in some of the work was composed of Dorking lime, sand, and Portland cement, which in the author's opinion had something to do with its failure. This lime and cement are very different in their action, the former hardens slowly, almost solely by the absorption of carbonic acid, the latter immediately (comparatively), by forming a crystalline double silicate of lime and alumina without the absorption of carbonic acid. The lime would certainly not attain any great hardness at the end of eight or ten days. If ground lime was used it would slake and expand some time after the cement had begun to set, consequently the mass would be quite disintegrated, and little better than sand; unless the proportion of cement was very small, in which case the particles of set cement would be separated and form a matrix, round which the lime mortar could form a film as with ordinary sand. Whether the lime was ground or previously reduced to powder by slaking, it is questionable if the cement might not give an excess of alumina to the compound, and thus render it liable to contraction, and the work to uneven settlement; but sufficient particulars are not at the author's disposal, to enable him to give an opinion with confidence, though those stated are to some extent borne out by personal experiment, and by the fact that, on the destruction of the chimney with gun-cotton a portion 5 feet in height was blown away, and every brick parted.

"Ah! you cannot make mortar and construct works as the Romans used to do!" is an exclamation with which those engaged in construction are more or less familiar. Nevertheless, it is an erroneous conclusion, formed by the com-

parison of their work with the flimsy structures put up by speculative builders in the cheapest and worst possible manner. But when their works are compared with such as those of Mr. John Fowler, whose "Metropolitan" brickwork cost more to pull down than it did to erect; and their mortar with such as that employed by the engineer to the Mersey Harbour Board, their doings sink into insignificance; for this mortar, when only six months old, sustains the same tensile strain as the highest result, recorded by Vicat, of tests with Roman mortar taken from the south of France, the average results of experiments with these two mortars being nearly two to one in favour of the former; and the author has been called upon to experiment and report on mortar which has, at the end of six months, sustained a tensile strain twice greater than that of the highest result above referred to, which was taken from the Tour Magna, and had centuries of age to its credit.

These remarks do not refer to Roman edifices, world renowned for their durability and æsthetical effect, which rely for their strength on the bulk and weight of the stones; but to structures in which "the minimum amount of material is made to perform the maximum amount of work."

Too much care cannot be exercised in ascertaining the proper proportions of lime, sand, and other ingredients. It is a peculiar circumstance, that the better and more hydraulic a lime, the less sand it will take up when made into mortar. Blue lias, among the most (if not the most) hydraulic and strongest natural limes to be found in England, like our artificial and natural cements, is completely spoilt by an excess of sand. To prove this bricks were bedded in the manner previously described, with blue lias and common greystone lime mortars, mixed in the proportion of 1 of slaked lime to 4 of sand, and at the end of three months required respectively 140 and 336 lbs. to tear them asunder. Had the

proportions of lime to sand been 1 to 2, the results would in all probability have been more than reversed. Blue lias is obtained in Somerset, Dorset, Glamorgan, and Warwickshire. That supplied by Messrs. Tatham, Kaye, and Co., of Paddington, is obtained in the last-named county. Its cost delivered in London, in December, 1873, was 25*s*. per ton when ground (one ton equals slightly less than one and a half cubic yards), in which state it is generally used, being, when in the lump, somewhat troublesome to slake; and unless the mortar is mixed in a mill, or thoroughly slaked, it is difficult to handle, as it will begin to set in ten or fifteen minutes—not a desirable property, unless in the case of hydraulic construction, where it is invaluable. The mill, keeping the mixture in motion, will do away with these very quick-setting properties, without in the least damaging its ultimate strength or hydraulic qualities. In recalling the author's early acquaintance with mortar, he well remembers his first attempt to slake blue lias; being informed that there would be some difficulty, and that the operation would take time, the lime was deluged with water every five or six hours. Six or eight of these drenchings having no effect, a lime-burner was applied to, who explained that blue lias requires but little water, and after being wetted it should be made into a heap and covered with sand. If previously ground, a good practice is to spread it under a shed some little time before it is required for use, and so slake it by the moisture of the atmosphere.

The brickwork of the Metropolitan and District Railways was constructed with mortar composed of this lime, ground in a mill for from fifteen to twenty minutes; and it is only fair to Messrs. Tatham, Kaye, and Co., who supplied the lime, to say, that in the destruction of a portion of the work a short time after its erection, the bricks as often broke through themselves as through the mortar joints.

In no material of construction is the ignorance of the *practical man* more manifest than in his treatment of mortar. He will often argue that since it is advisable to sour lime for the purpose of the interior decoration of houses, that therefore it should be so treated for all other work; forgetting, or not knowing, that by this means its indurating properties are much, if not quite, impaired; but had he the sense to know and admit his ignorance, he would only prove himself to be little behind the most experienced and learned men on the subject; for sound conclusions cannot be formed without analysing the properties of the lime, sand, and other materials of which the mortar is composed. The study of the question in this manner yielding no immediate returns, is quite beneath his consideration.

Whether lime is to be used for mortar or concrete, it should, in the author's opinion, have time to slake before being put in place.

It must always be borne in mind that "the strength of a chain is in its weakest link," so, also, the strength of brickwork subjected to tensile strains is in its mortar.

135, George Street, Edinburgh,
March 16, 1874.

Discussion.

"Continental mortars made with rich limes mixed artificially with pozzuolana, were stated to answer well in many large works; and it was suggested that, when to be obtained cheaply, it should be used."

Author's Reply.—Without doubt pozzuolana adds to the strength and gives hydraulic properties to rich limes; but nothing can be gained by its admixture with a really good hydraulic lime.

Artificial pozzuolanas were much used by the Romans, at a later date by the French, and now pulverized brickdust forms a regular article of commerce in Cuba; it is well mixed in a dry state with the lime and sand, in the proportions, 1 brickdust, 1 lime, and 2 sand; this is said to be a good substitute for cement. In the Roman and French works the mixture of artificial pozzuolana with the lime was found to answer well in all constructions on dry land, or in contact with fresh water; but, according to Vicat, artificial pozzuolana should never be used for sea works, as, owing to the presence of magnesia in salt water, a secondary chemical action will be set up, which will ultimately reduce to powder that which would for some time pass for first-class mortar. In using brickdust, care must be taken to have it very finely pulverized, and thoroughly mixed with the lime and sand, and too large a proportion must not be employed.

A more expensive, but preferable way, of employing artificial pozzuolana is to mix the clay with slaked lime, in the proportion of 1 of the former to from 3 to 5 of the latter, according to the amount of alumina and silica already in the lime, and to calcine the compound, which may then be packed ready for use. By proceeding in this manner the materials are intimately mixed, and any action

of one ingredient on another takes place previous to the complete setting of the mortar; as is the case with good Portland cement, which is made in a very similar manner to the above, the difference being that the hydrate of lime is replaced by chalk, that is, nearly pure carbonate of lime, and a large quantity of water is employed in the process of mixing.

"The Chairman remarked that he did not think the admixture of ashes would greatly assist in the ultimate turning of mortar into 'stane,' as conveyed in the rhyme."

Author's Reply.—It is not probable that the mortar will ultimately attain to any greater hardness by the addition of ashes; in fact, if used in too large a quantity, they will absorb the water, and, consequently, the strength of the mortar will be much impaired; that is, if the mortar be not thoroughly well ground with water in a mill.

On the Mersey Dock Estate, ashes, being plentiful, are employed, not with a view of attaining greater ultimate strength, but in order to produce an inexpensive and quicker drying, if not setting, mortar; which is used in brickwork where speed in execution is generally an important consideration.

"Exception was taken to the stress laid on the question of tensile strains in brickwork; stone and brick structures being considered not to be often in that state."

Author's Reply.—Hundreds of instances are at hand to prove that mortar does exert in a structure a very considerable tensile strength; take, for instance, the St. Rollox chimney, 435′ 6″ in height; did the mortar not exercise any tensile strength, the limiting condition of safety would be when the *centre of pressure* was within the middle half of the diameter; but it is well known that in a high wind the top of this chimney sways very considerably, and the limits of deviation in the position of the *centre of pressure* must frequently be very much exceeded.

Were it not for the tensile strength of brickwork, a very different style of architecture to that now existing in London and other places would be witnessed. Many of the arches are simply fictitious, being in tension instead of compression, as the form would seem to indicate. Numerous buildings must necessarily have come down if this were not the case, for two-thirds of the arches are entirely without abutments.

"Portland cement mortar was stated to be now exclusively used by some engineers."

Author's Reply.—The exclusive use of Portland cement mortar can only indicate ignorance of the qualities of many natural hydraulic limes, and this want of knowledge is dearly paid for. Portland cement mortar, of equal strength with blue lias and other hydraulic mortars mixed in the proportion of 1 of lime to 2 of sand, will cost little less than double the price, and give very slight compensating advantages. For although the quality of limestone may vary from the same quarry, there can be little doubt but that satisfactory results will be obtained, if as much care is exercised in selecting the lime as is requisite in order to secure good Portland cement.

"Grinding mortar in a mill was stated to be anything but beneficial to its quality, as in obtaining law evidence for an important case it was found that sand, ground five minutes, was reduced to mud."

Author's Reply.—No doubt the sharpness of the sand will be impaired by grinding; but the author cannot conceive how sand can be reduced to mud, unless the pan was dirty, or the sand of a very loamy nature and crushed under the rollers, which in that case must have been, as they never should be, without vertical play. In order to ascertain the proper time for grinding mortar, experiments were made with mortar ground during periods of varying length, and it was found

when the pan had made about 600 revolutions that the mortar was at its best, which at twenty revolutions a minute will be half an hour, or the time employed on the Mersey Dock Estate. The tensile strength of mortar, ground from 300 to 400 revolutions, or from fifteen to twenty minutes, will fall short of the above by about 20 per cent. Cement mortar should never be ground.

"The following was put forward as a good practical method of determining the proportion of lime to be used in mortar or concrete. Fill a box or bucket with the sand to be used, and, after well shaking it down, pour in water until the sand is thoroughly saturated, the quantity of water thus added will be the amount of lime which should be used. Or, in the case of concrete, fill the box with screened ballast, shake in as much sand as possible, and add water until the whole is saturated, the quantities of sand and water will respectively equal the proper proportions of sand and lime to be put into the concrete."

Author's Reply.—Although this is usually accepted by the profession as a truism, still the author has grave doubts as to its correctness. If accepted as correct, all difficulty in ascertaining the proper proportions of sand and lime is at once removed, and for the same sand the quantity of lime will remain constant, whatever be its qualities or characteristics, consequently the remarks on blue lias and the experiments of Pasley, Vicat, as well as those of the author, are rendered unreliable; hardly admissible when it is an acknowledged fact that the more hydraulic limes take up less sand than the rich ones when made into mortar. Vicat found that the voids in sand, the grains of which were one hundredth of an inch in diameter, amounted to 33 per cent. of the whole bulk; therefore, if the water theory hold good, mortar made with sand of this class will have to be mixed in the proportion of 1 to 3. Supposing a hydraulic mortar is required, nothing

prohibits the use of Roman cement to obtain the desired hydraulic properties, Roman cement being simply an eminently hydraulic natural lime; unfortunately the resulting compound will be anything but satisfactory; if greystone lime be substituted for the Roman cement, a very good ordinary mortar will be produced, and if hydraulic properties are left out of the question, it will surpass in every respect the other much more costly mixture.

The amount of sand to be allowed in gravel for making concrete may be ascertained by the means mentioned. Gravel and sand when wetted decrease in volume; the latter is stated by Pasley to occupy only four-fifths of the space which it did when dry; the lime and water, adding very little to the volume, will about make up for this decrease, and so the resulting quantity of concrete will approximately equal that of the original gravel.

"The comparisons in regard to Roman mortar were thought to be incorrect."

Author's Reply.—The high repute in which Roman mortars are held is probably due to people who, after poking them about with their sticks, return home to find next door a "jerry" house run up with mortar composed of lime slaked and mixed some six months previously, consequently erroneous opinions are formed. So far as the author is aware, there are no data recorded which show the superiority of Roman over good modern mortar. On the contrary, by visiting the works themselves, it will be found that the Roman mortars, although frequently composed of good materials, are generally very badly mixed, and give when tested the poor results previously mentioned. There is one thing which may possibly tell against them; in testing the Liverpool Dock mortar, it was moulded, as is usually the case, into the required form when soft, whereas the Roman mortars were cut from the solid, and, as is well known,

mortar is not a material to withstand manipulation with edge tools.

Author's concluding Remarks.—The practical nature of the foregoing may be inferred from these remarks. Four indicated H P are given as necessary to drive the mortar mill described, which, although differing slightly in detail, is similar in dimensions and general principle to those used on engineering work. If the mill is driven by a good ordinary portable engine, it will consume in every-day practice for each I H P 5 lbs. of coal per hour, a high allowance when compared with agricultural show trials, and other experimental attainments, where the same duty is performed by from $2\frac{1}{2}$ lbs. to $3\frac{1}{2}$ lbs. of coal, and in well-designed compound engines with even less. In these instances, by care and constant attention, all the contingencies of practice are guarded against; actual work is a different thing; among other causes of high consumption of fuel may be mentioned, steam running to waste at the safety valves, boilers and engines worked below their power, and, through inattention or ignorance in stoking, unconsumed coal going up the chimney, only to pollute the atmosphere, in either visible and unpleasant smoke, or in the invisible form of carbonic oxide, which is even worse than black smoke; a smokeless chimney is not therefore to be always held as harmless. Intending patentees would do well to bear in mind that smoke once formed cannot be burnt in the furnace; it can only be converted, without effecting any economy of fuel, into the invisible negative poison just named. Smoke burning does not certainly come under the head of "mortar;" still, little harm can result from these few words. With the assumed consumption of 5 lbs. of coal, 4 I H P will require 200 lbs. per day of ten hours, or, roughly, 2 cwt., which at 20*s.* per ton is 2*s.* a day. One man can very well look after an engine capable of exerting 40 I H P, the

amount to be charged to each mill will therefore be one-tenth of his wages, which at 6*s.* per day will be 7*d.*; the wear and tear of engines and mills at 10 per cent. on their total value will amount to 6*d.* One superintendent can look after at least the mortar mixed in ten mills, and if he be paid 10*s.* per day, 1*s.* must be charged to each. With the wages of one labourer for attending the mill, as in paper, at 3*s.* 6*d.* a day, the estimate for machinery and labour will be—

	s.	*d.*
Coal	2	0
Engine man	0	7
Wear and tear, 10 per cent.	0	6
Superintendence	1	0
Labourer	3	6
	7	7 per day.

If the mortar be ground half an hour, each mill will make twenty turn-outs of one quarter of a yard, or in all 5 yards per day, at a cost of 1*s.* 6*d.* per cubic yard. On contracts mortar is seldom ground more than twenty minutes; in this case the amount turned out per day will be 7½ yards, costing 1*s.* per cubic yard, an amount certainly not sufficiently large to sanction an engineer doing away with grinding on the score of economy. The estimate will not hold good for a less time than twenty minutes, as 7½ cubic yards per day is about as much as one man can properly look after; if a less time is calculated upon, a portion of another man's time must be added.

Dorking lime is stated in paper to cost 13*s.* per cubic yard; and as it augments in slaking about two and a half times in volume, its cost, with 10*d.* per yard added for wheeling and slaking, will be 6*s.* per cubic yard of slaked lime.

If the mortar be ground twenty minutes, and the propor-

tions be 1 of lime to 2 of sand, its cost will be somewhat as under:

	s.	d.
Labour and machinery in mixing	1	0
One cubic yard of sand	3	0
Half a cubic yard of slaked lime	3	0
Owing to contraction of materials, add to make up quantity	0	6
Per cub: yard of mortar	7	6

It is further given that a yard of brickwork requires a quarter of a yard of mortar, which, of the above description, will cost 1*s.* 10½*d.*; there will be 380 bricks at 30*s.* per thousand = 11*s.* 5*d.* A bricklayer and his labourer will, if well looked after, build about 3 cubic yards of brickwork per day; and supposing their joint wages to be 9*s.*, the cost per yard for construction will be 3*s.*, the estimate for brickwork will then be:

	s.	d.	
Mortar	1	10½	
Bricks	11	5	
Labour	3	0	
Scaffolding, &c.	0	8½	
All superintendence	1	6	
	18	6	per cub: yard.

These estimates, which do not include either contractor's or sub-contractor's profits, will necessarily vary with the labour market and the cost of materials; still there are some items, the quantities of which will remain constant under most circumstances.

PRACTICAL IRONWORK.

Being a Paper read at a Supplemental Meeting of the Institution of Civil Engineers. April 24th, 1874. George Robert Stephenson, Esq., *Vice-President, in the Chair.*

In offering the following remarks on "Practical Ironwork" to the consideration of a meeting of members of one or more grades of this great institution, of which the author is proud to be a student, no time will be wasted by prefatory detail; but as all present are more or less familiar with the subject, it will simply be assumed that the skeleton of an iron structure has been decided upon, and the strains ascertained to which each of its members is to be subjected; and the purpose of this paper will be to investigate the practical considerations affecting its design, strength, and construction.

No attempt will therefore be made to delineate the process by which these strains have been arrived at, as that portion of the subject can only be mastered by deep and patient study of existing structures and standard works on the subject.

In apportioning the proper amount of metal to meet the strain, the first care should be to employ as nearly as possible only iron of the economical sections, weights, and lengths to be obtained in the iron market; points not unfrequently overlooked.

For instance, angle-irons $4'' \times 4'' \times \frac{1}{2}''$ can be obtained in

lengths of 30 feet without extra cost, and they are sometimes rolled at an additional cost to 40 feet in length. At the Vienna Exhibition a Z section of rolled iron was exhibited 85 feet in length.

Angles, tees, and similar sections can be obtained in moderate lengths without extra charge, when the sum of the sides does not exceed 8 or 9 inches; with larger sections, such as H iron, having a depth of about 12 inches, the cost will be something over 20 per cent. more than that for tee and angle-iron complying with the above condition.

In the same way an extra price is generally paid for bars exceeding 6 inches, although up to 8 or 9 inches it is not excessive, and they can be obtained from some manufacturers up to 12 inches in width.

It is in dealing with plates that the term *extra* must always be kept in view, as by bad designing the cost of the iron may be nearly doubled; a plate to cost the ordinary market price must be under 4 cwt. in weight, 15 feet in length, 4 feet in width, and must not be of irregular shape.

In order to better illustrate this point, a plate 15′ 0″ × 5′ 1″ × $\frac{3}{8}$″ may be taken as an example, and the extra charges, according to formal tender made in 1873 by a first-class firm for "best best" crown boiler plates, will be:

For weight	£12	0	per ton.
„ length	0	0	„
„ width	3	10	„
„ cutting	0	0	„
In all	£15	10	per ton.

The price for iron by this tender, complying with the previously mentioned conditions, was 19*l.* 10*s.*, to which this extra charge must be added, bringing it up to 35*l.* per ton, or nearly double the cost of ordinary plates.

Although not in the original paper, it may be well to here introduce a few additional figures from this Staffordshire tender.

LONDON, E.C., *August* 19*th*, 1873.

The ________________ Company.

GENTLEMEN,

Our present prices of Crown Boiler Plates are as under, viz. for ordinary sizes—

"Best"	£18	10	per ton.
"Best Best"	19	10	„
"Best Best Best"	21	10	„

Terms, monthly and cash, less 2½ per cent.

Delivered free to your Works, with EXTRAS, as per List:

For Width.

Plates over 4 ft. and not exceeding 4 ft. 6 in., 30*s*. per ton.
„ „ 4 ft. 6 in. „ „ 5 ft. 60*s*. „
with 10*s*. per ton more for each inch over 5 ft. wide.

For Weight.

Plates over	4	but not exceeding	5 cwt.,	each	20*s*.	per ton.
„ „	5	„ „	6 cwt.	„	40*s*.	„
„ „	6	„ „	7 cwt.	„	70*s*.	„
„ „	7	„ „	8 cwt.	„	100*s*.	„
„ „	8	„ „	9 cwt.	„	140*s*.	„
„ „	9	„ „	10 cwt.	„	180*s*.	„
„ „	10	„ „	11 cwt.	„	240*s*.	„

For Length.

Plates over 15 ft. and not exceeding 20 ft., 30*s*. per ton.
„ „ 20 ft. „ „ 25 ft., 60*s*. „

For Sketches.

Plates cut to irregular shapes .. .. .. 30*s*. per ton.

These considerations must be attended to in designing buckled, corrugated, or other manufactured forms of plates, as if they require to be made from iron on which extras are charged, their cost will necessarily be enhanced.

At the Vienna Exhibition, plates were exhibited 52′ 6″ × 5′ 3″ × ·63″, and 48′ 7″ × 4′ 2″ × 1·23 inches, weighing 3 and $4\frac{1}{2}$ tons respectively; these were of course exceptional and very costly plates.

The extras for plates do not become very considerable until a weight of 7 cwt. is exceeded, and a width of 4′ 6″; but when a width of 5 feet is reached, the extra is charged per inch; and by the tender before referred to, it amounts to 10*s.* per inch per ton, in addition to 3*l.* charged extra for a width of 5 feet.

A long plate becomes expensive on account of the difficulty in handling it, and the great care required to be exercised in putting it through the rolls, in order to avoid making it of irregular shape, in which case there is a large amount of waste in cutting the plate to the required length and area; and sometimes it becomes what is technically termed a *waster*, which has to be laid aside; to cover these contingencies much larger blooms have to be made.

Contractors have sometimes to tender for ironwork when the designs contain many of these irregular lengths and dimensions; it then becomes necessary for them to estimate the probability of the conditions of the design being enforced, not always an easy matter; but when the reason for the employment of *extra* iron is obvious, they tender accordingly. Occasionally designs exhibiting a total ignorance of ironwork come into the contractor's hands, in which case estimating becomes very difficult, and to be on the safe side a large percentage must be put on to cover contingencies.

The contractor can generally estimate the quality of the work which will suffice, by the workmanlike style, or otherwise, in which the drawings and specification are made out.

Angle-iron bends and joggles should be as few and simple as possible, and bent or cut plates should be few and far between.

Tee stiffeners to the web of a plate girder, if splayed out clear of the angle-irons, will cost about half as much for smithing as when required to fit close round them, and will answer their purpose equally well. Where the flanges are too narrow to admit of four rows of rivets being put through at one cross section, this form of stiffener cannot be used; where the flange requires only two rows of rivets, but is of sufficient width to admit of two additional rivets, the stiffeners can be generally so arranged that these rivets shall in no way affect the strength of the girder; as, owing to the unvarying thickness of plates, there is always an excess of strength in the flanges of a plate girder, at points not very far distant, and it is here that the stiffeners should be placed.

The harassing little, but still most important, rivets should be carefully considered, as it is on them that the whole strength of the structure depends; no amount of extra metal in the body of a tie can compensate for a badly-constructed joint. The author would advise careful consideration of the position, diameter, and pitch of every rivet, previous to placing the design before contractors; as carelessness or ignorance on this point at once indicates the amateur designer, who is often led astray by the facility with which he can ornament his drawings with these little circles or red crosses.

Drawings of ironwork should be as complete as it is possible to make them, free from all ambiguity, leaving no room for doubt as to what is intended; in order to effect which dotted lines must largely be employed; it is impossible to confuse a drawing by putting dotted lines in their proper places, as it must be borne in mind that the drawings are not to be submitted to a parliamentary committee, but to men accustomed to most complicated mechanical drawings. A working drawing is not necessarily pretty; it should be neatly and accurately made, and should require no explanation whatever.

The pitch and diameter of the rivets should in all cases be marked upon the drawings, and their relative positions in the structure should be pointed out. In the construction of a bridge, it is generally necessary to draw the whole, or parts of it, full size, with its required curve or camber, in order to ascertain the accurate lengths of its separate portions, and the exact relative positions of the rivets, which will by this means be very accurately ascertained; these positions will then be marked on a template, from which any number of similar plates can be made.

It is usual to specify that when one portion of a structure is to be riveted to another, the holes must be made with such accuracy, that a template with studs fixed in the relative position of the rivets, one-sixteenth less than them in diameter, shall enter all the holes at the same time, and go completely through the various thicknesses of plates. If the holes are not fair to one another, the shearing area of the rivet is reduced, and therefore the joint will not be of its calculated strength, to cover which fault it is advisable, in calculating the number of rivets, to consider their diameter one-sixteenth less than that shown on the drawings. All rivets should have a good and sufficient head, and when inspecting the construction of ironwork this point should be enforced.

The strength of rivet-iron will be considered hereafter; but a good practical test is that a rivet shall be hammered when hot to less than one-eighth of an inch in thickness, without showing any signs of cracking.

Although a difference of opinion exists among engineers on the question of drilled *versus* punched holes, it would appear from the following experiments, carried out by Mr. Cochrane, and kept on record in the 'Minutes,' vol. xxx., that practically there is little to be gained, so far as the strength of plates of good quality is concerned, by using one method more than the other.

A bar of Lowmoor 2 inches × $\frac{1}{2}$ an inch, with a drilled hole $\frac{7}{8}$ths of an inch diameter put through it, sustained 235 cwt., and with a punched hole, 226 cwt. With bars of hard crystalline iron, of the same dimensions, that with the drilled hole sustained 250 cwt., and the punched hole 244 cwt. With bars each with three holes, one drilled, one punched, and one punched $\frac{3}{4}$-inch and drilled to $\frac{7}{8}$ths of an inch diameter, in all cases they broke through the punched hole; which would be inferred from the above results, but which are still well within the limits of variation of the breaking weights of our best irons.

When we leave strength out of the question, and consider only workmanship, the method to be adopted must be left to the judgment of the engineer. If a number of plates are to be riveted together, or an intricate junction made, unless great care is exercised by a very experienced hand at a punching-machine, the drilled holes will be more accurate; but when a simple plate and angle-iron are to be joined together, the balance of strength in favour of drilling is not sufficient, in the author's opinion, to sanction the extra cost.

A want of proper welding in the lamina of iron may be sometimes detected when punching; it is difficult to punch near the edge of bad iron, but a drill may be put through almost any rubbish.

The cost of cast iron may be also greatly enhanced by bad designing; but as civil engineers have not often to deal with very intricate forms, it may be sufficient to say that the molten metal is run into moulds, and that, with the exception of very large castings, which are built up bodily, these moulds are formed by placing in the sand a pattern in wood of the object to be cast. Of course this is well known to all (?) but still not always remembered, if we may judge from the number of "boxes" and "cores" sometimes required for what ought to be simple castings.

In a casting such as a transverse truss of an arch bridge, the different portions of the truss should have nearly the same amount of cross section; as, if this is not attended to, the members with the smaller body of metal will cool first, and the remaining portions in cooling will put an initial strain on the iron, which may distort or even cause its destruction; for these reasons every precaution should be taken to insure that the casting may cool evenly.

As regards the quality of cast iron, for want of space a few words must suffice. For structural purposes it should be melted at least twice, and mixtures should be made which will stand the following test: a bar 3′ 6″ × 2 inches × 1 inch run each day from the cupola from which the castings are being made, should, when placed edgeways on 3 feet bearings, sustain a load applied at the centre of 28 to 30 cwt., and give before breaking a deflection of $\frac{5}{16}$ths. of an inch.

The cast iron of the sleepers for the Great Indian Peninsular Railway had to undergo tests very similar to this, and, in addition, the sleepers had to stand a weight of $3\frac{3}{4}$ cwt. falling a height of 5′ 6″, the same having previously been subjected to blows from this weight falling 2′ 0″, 2′ 6″, 3′ 0″, &c., up to 5′ 6″, being at the time of testing on sand not more than 24 inches deep; the average height required to break the sleepers was 7′ 9″. The cast iron was also to stand a tensile strain of $11\frac{1}{2}$ tons per square inch; and the average was 13·07 tons—a very high but obtainable result.

It is not advisable to restrict the manufacturer by stating the proportions and classes of iron to be mixed, as he will in most cases be better able to judge of the mixtures to be made in order to produce a specified result. Not that the author would advocate leaving too much to the practical man; as we not unfrequently come across the "miscalled practical man," who will tell you that he is able to judge of iron by its appearance, a statement which must be accepted with every

caution, as men who have given much more time than most engineers can devote to the study of iron, very often err in their conclusions formed only by inspection. An instance came under the author's notice where a fracture of rod iron was shown, and pronounced first-class, and on the other end of this rod, not six inches in length, being exhibited, it was pronounced little better than trash; in the first instance the fracture was of a fine silky fibrous nature, which denotes that the iron is soft, tenacious, and generally of good quality; whilst in the latter case it presented a rather coarse crystalline fracture, which, owing to the largeness of the crystals, showed that the iron was not quite of an uniform quality; but the appearance was chiefly owing to the mode of testing, as, if iron is broken by a sudden strain, it invariably presents the crystalline fracture; whereas, if good iron, and broken by a gradually increasing strain, the fibrous fracture always results; nevertheless, it is the generally accepted opinion that the fibrous fracture cannot be obtained, by any mode of testing, from really bad iron; the gradually increasing strain should, therefore, be applied, and the fibrous fractures must result from good iron, more especially from rolled iron or plates.

Whether the iron is fibrous or crystalline, its quality may to some extent be judged by the fineness of its texture, and the irregularity of its fracture.

Ironmasters are to some extent averse to testing; the author was advised by one of these gentlemen, of high standing, to exhibit his knowledge of the subject by simply specifying "best merchantable iron," and if from inspection it was found not to be good, it could then be tested; but after what has been said, it is needless to point out the impropriety of such a proceeding. It was also hinted that tests were instituted to exhibit the cleverness of engineers, instead of which they simply exposed ignorance; and, no doubt, there is often room for such remarks.

According to Mr. Edwin Clarke, the permanent set of a bar 10 feet long and 1 square inch in section, was for 3 tons' strain $\frac{1}{468750}$th, and at 10 tons $\frac{1}{74953}$rd of its length. The stretch of iron up to 12 tons is to some extent uniform, and may be taken at $\frac{1}{10000}$th of its length for each ton; but beyond this strain it becomes irregular, which accounts for the ultimate deflection of a beam being almost beyond calculation.

In the early days of engineering it was a common practice to make things strong enough, on the erroneous supposition that a structure tested with something more than its working load would be quite safe. This may or may not be true, according to whether the *limit of elasticity* of the iron is exceeded; if such be the case, the ultimate destruction of the structure is only a question of time; under any circumstances the design of a structure on such an hypothesis is now quite inadmissible, owing to the increased cost of labour and materials, and the criticism to which a design is always subjected.

The *limit of elasticity* of iron is that point at which the same strain will produce an increasing permanent set. An example will, perhaps, more clearly express this important point: if a bar be tested with a load of 8 tons, and the permanent set not increased by any number of repeated applications of this load, 8 tons is said to be within *the limit of elasticity of the iron;* if, on the other hand, the permanent set increases ever so little, the limit of elasticity has been exceeded.

It would, therefore, be well to specify that iron shall not take an increasing set with 8 or 9 tons, but that with 11 or 12 tons it must show a decided permanent set over and above that obtained with 8 tons; the latter condition is required, in order to ensure the first being complied with, as an unprincipled manufacturer might put a higher strain than

8 or 9 tons, and so take all the stretch out of the iron previous to its official testing, in which case any iron would comply with this condition. This point is, in the author's opinion, one of great importance; for, supposing the working strain put upon the iron to be 5 tons to the inch, although this may be only $\frac{1}{4}$th of the ultimate breaking strain, the factor of safety will only be 1·6 in place of 4, as might appear at first sight; that is, assuming the limit of elasticity to be 8 tons.

The working strain to which iron is to be subjected is governed by the specified quality and tests which it is to comply with. In moderately small structures, where the working load is comparatively great in comparison with the weight of the structure, it will be found in most cases more economical to employ an ordinary quality of iron with a small working strain, than to use an exceptionally good material. The working strain of the iron is also governed by the character of the load. In the case of a girder supporting the wall of a building, a strain of $6\frac{1}{2}$ tons might with safety be put upon good ordinary iron; but in a small railway bridge of about 20 feet span, where the moving load is to a great extent indeterminable, it is not well to impose a greater strain than 4 or $4\frac{1}{4}$ tons.

From the author's experience, and a careful study of the many results obtained by such high authorities as Mr. Kirkaldy, he concludes that ordinary iron to be found in the market should comply with the following conditions:

Rod, bar, and rivet iron should sustain an ultimate tensile strain of 24 tons per square inch, elongate 15 per cent. before breaking, and be reduced in sectional area about 25 per cent.

Angle-iron, an ultimate tensile strain of 22 tons, elongate 9 per cent., and be reduced in sectional area about 15 per cent.

Plates, with the fibre, an ultimate tensile strain of 20 tons, elongate 6 per cent., and be reduced in sectional area about

8 per cent.; and across the fibre, 18 tons, elongate 3 per cent., and be reduced in sectional area about 5 per cent.

These conditions, in a tabulated form:

Description of Iron.	Breaking Strain per sq. in.	Elongation.	Contraction of Area.
	tons.	per cent.	per cent.
Rod, bar, and rivet..	24	15	25
Angles, tees, &c.	22	9	15
Plates, with fibre	20	6	8
Plates, across fibre	18	3	5

These requirements are not at all excessive, and should, therefore, not enhance the price of ordinary Staffordshire beyond the insignificant sum for testing purposes.

The samples on which these investigations are to be carried out, ought to be from 8 to 10 inches in length between the points of attachment.

The toughness and ductility of the iron can be estimated either from the contraction of area or elongation; it is unnecessary, therefore, to investigate both of these points, the choice between them being decided by the facility with which they can be ascertained; but where the experiments are to be carried out at testing works, such as those of Mr. Kirkaldy, it is immaterial which is specified, as, by experience, either may be ascertained with equal precision.

No doubt the carrying out of tests will involve the expense of continuous inspection; but the engineer, by rigorously enforcing his specification, will not only gain the esteem of every honest contractor, but will have the satisfaction of knowing that he is obtaining good materials and workmanship.

On the completion of the structure it should be tested with a load slightly greater than its working load, and the deflection noted.

This test of workmanship must only be taken in conjunc-

tion with the test of materials, as a joint or portion of the structure might be on the point of breaking, and yet not stretch sufficiently to materially alter the deflection; and hard brittle iron under the deflection test may appear superior to a better and more desirable quality.

If an iron structure is well designed, erected, and taken care of, the author can see no reason why the term "life of ironwork" should not become a thing of the past; as iron imbedded in concrete has been found, at the expiration of twenty years, as free from decay as on the day of its manufacture.

A few words on the preservation of ironwork will, therefore, perhaps not be out of place. It is advisable to coat a casting with oil, paint, or tar, as soon as it leaves the sand, as by so doing the hard outer skin is preserved; and when in position it should be again coated. When cast iron is to be covered up, a preparation of coal-tar and naphtha is found to answer well; this may also be used to advantage for the upper surface of buckled plates, or other forms of road platforms.

Wrought iron, when hot from the rolls, is sometimes brushed over or dipped into boiled linseed oil; but a better and less expensive way, in the author's opinion, would be to brush it over with boiling hot oil when cut to something like its permanent shape, and after its workshop manipulation to again treat it with oil. Previous to being erected, any abutting surfaces should be also coated with red-lead, or other metallic paint. A preferable way, if thoroughly carried out, would be to let the iron slightly rust, and have it well scraped before its treatment with oil, as by so doing the oxides or blue shales, which in a more or less degree are inseparable from its manufacture, would be removed; they are very apt to peel off after a short time, and bring the paint with them—more especially in a structure subjected to vibration. This is not of such vital importance for home work under constant supervision; but for foreign up-country work too much care

cannot be taken to preserve the surface of the iron from atmospheric and other influences.

All work should be so designed to allow of its being painted, and plates less than a quarter inch in thickness should never be used.

The linseed oil brushed over the iron, as before pointed out, will form a kind of varnish, and is an excellent preparation for the after coats of red-lead or other metallic paint, of which not less than two coats should be used, the last being of approved colour and put on after the work is erected in place.

On account of the great cost of red-lead other metallic paints are now generally substituted. That expensive ingredient was often replaced by coloured earths, which were mixed with inferior oils, and it is needless to state that the quality of the paint is to a great extent governed by the oils with which it is made.

All tooled surfaces should be well covered with tallow and white-lead.

Those wishing to become acquainted with the practical items of ironwork cannot do better than study 'Works in Iron,' by Mathieson; and 'Experiments on Wrought Iron and Steel,' by Kirkaldy. But a year devoted to the vice and bench, in a good ironwork or general mechanical shop, will be of more service than any amount of reading; and we have it on no less authority than Mr. John Fowler, that a mechanical training is absolutely essential to those aspiring to any eminence as civil engineers.

In conclusion, the author trusts his confrères exonerate him from any desire or attempt to exceed his province as a "student" by the terms which he has found it necessary to employ in his treatment of "Practical Ironwork."

135, George Street, Edinburgh,
March 23, 1874.

Addenda.

At the meeting samples of iron were produced, some of which exhibited different classes of material rolled together in the same bar; a piece of rod iron $1\frac{1}{4}''$ diameter, to all appearance of uniform quality, when struck over an anvil flew to pieces on one side, disclosing the fact that one half was little better than cast iron, whilst the other seemed to be fibrous and good; from which it is seen that even in iron of moderate dimensions very different qualities may be combined. Many pieces of rivet iron, picked at random by the author from the stores of the Thames Iron Works, bent double without showing any signs of cracking.

The opinion has been advanced that the tests advised in the paper are of too minute a character for actual practice; this cannot be the case, as similar ones are daily required by the Indian Public Works Department of Her Majesty's Government, as also by many private engineers. In the event of appliances for making such tests not being at hand, it is only necessary to specify that on the contractor receiving a quantity of iron into his yard, or at each separate delivery of iron from his own mills, he shall give notice to the engineer before proceeding to use it in any of the work; whereupon two or three samples are selected from this quantity of perhaps 8 or 12 tons, and put into a box, addressed to David Kirkaldy, Esq., The Grove Testing Works, Southwark. After a lapse of a few days a report will be returned containing all particulars. When it is considered that "iron is not always iron, for sometimes it is simply rubbish," tests are absolutely necessary to enable an engineer to state with confidence what is the safe working load of a structure; and when dealing with constructions subject to much vibration, or great changes of stress, iron, like Port-

land cement, becomes a dangerous material in the hands of those ignorant of its qualities.

These remarks are made without taking into consideration the very improbable case of a structure made solely of Lowmoor, Bowling, Farnley, or other of the best Yorkshire irons, as the high prices charged for these prohibit their use except in positions requiring much bending or complicated forging during construction. In making a selection from iron bearing the above names there is little room for choice, all being practically equally strong and tough. This class of iron is seldom if ever tested by the buyer, the high price paid being sufficient to enable the manufacturer to carefully watch the iron in its progress from the pig through the rolls to the shears, rejecting at each step anything of at all doubtful character, so ensuring the highest standard, and consequently uniformity of quality. It is the commoner irons which exhibit great variations in strength and toughness.

The following disjointed extracts from the report of Mr. Lavington E. Fletcher, chief engineer to the Manchester Steam Users Association, on the Blackburn boiler explosion, will in a manner substantiate the remarks in reference to elongation, and contracting of area of iron when fractured by a tensile strain.

"Tensile strength, however, is not the only quality required in boiler plates. Ductility is necessary also. This is too often lost sight of. A plate with a moderate tensile power, if ductile, is better for boiler purposes, than with a high tensile strain if short and brittle. To ascertain the ductility of the plates under consideration reference may be made to the reduction in area at the point of fracture given in the first of the preceding tables.* It will be seen that this reduction in area varies considerably in different experiments. In the case

* For tables and full report, see 'Engineering,' April 24th and May 1st, 1874.

of the test plates cut from the top of No. 2 boiler, it is as low as 5·97, while the mean of the tests of the plates cut from the top of that boiler is 7·29. This does not indicate a high ductile power. The same inequality will be seen in the rate of elongation, which in one case was 1·97 per cent., in another 7·60 per cent."

"The result of the investigation of the quality of the plates is, that though adequate in tensile power, they had not, at all events some of them had not, that amount of ductility which it is desirable they should have had, seeing they were to be employed in the construction of a boiler to be worked at so high a pressure as 80 lbs. on the square inch. I do not consider, however, that the want of ductility was by itself the cause of the explosion."

The limit of elasticity defined in the paper is somewhat different from that given by most authorities, who state it to be the strain which produces a permanent change in form, that is, in ironwork, permanent set, which, in a bar strained with three tons per square inch, is $\frac{1}{468750}$ of its length;* it follows, if the above is a right definition, that this strain exceeds the limit of elasticity; and with the same iron any lower strain will produce a permanent set, provided sufficiently accurate instruments are used for its detection. It thus becomes a variable quantity, of no practical use whatever, being solely dependent on the adjustment of instruments.

In order that there may be no misconception of what the author considers as the true and practical limit of elasticity, the following elementary statement is made:

If a bar of iron be subjected to a strain of 10 tons, it will stretch a certain amount, and on the removal of the 10 tons it will attain nearly its former length; the difference between this length and its original length is termed its permanent

* See Paper, p. 50.

set. Now, if the 10 tons be again put on, and the bar at its removal does not go back to the length which it did before, that is, the original length plus the permanent set obtained in the first instance, the load of 10 tons exceeds the limit of elasticity. Again, if a strain of 8 tons be put upon a bar of similar quality and dimensions, and the permanent set noted; if, on the removal of the 8 tons after a second application, the bar goes back to its original length, plus the permanent set obtained in the first instance, the limit of elasticity has not been exceeded; so that, for the iron under consideration, it must be somewhere between the strains of 8 and 10 tons per unit of area. For good ordinary iron it is from $8\frac{1}{2}$ to 10 tons to the square inch.

Therefore, if a working strain be put upon the iron which will cause the permanent set to increase ever so little at each application, by repeated applications the bar will continue to elongate, and the structure of which it forms a portion must ultimately fail, and, as stated in the author's paper, "the destruction of the structure is only a question of time;" it may be one or one hundred years.

Assuming the limit of elasticity to be 8 tons, and the working strain 5 tons per square inch, taking time into consideration there will only be a margin of safety of 3 tons; dividing the maximum limit 8 tons by the working strain 5 tons, the factor of safety of 1·6 is obtained in place of 4 found by dividing the ultimate breaking strain of 20 tons by 5 tons as is commonly done. Therefore, a structure which is four times too strong with a dead load, may only have a factor of safety of 1·6 when subjected to a moving load of the same amount.

The working strain of iron is generally taken low enough to be within the limit of elasticity, or to exceed it by so little that it takes a very long time to cause the destruction of a structure in the manner described, and when it does take

place the work is simply, but quite truly, considered as worn out; still, by proper designing, wearing out in this manner ought to be nearly impossible.

The foregoing is quite consistent with practice; on the wearing out of a railway bridge it is usually found that the cross girders are first destroyed. Main girders exceeding a span of 50 feet, well designed and constructed, need never fail if kept properly painted. Small main girders, in railway practice designed on the ordinary assumption of 16 tons on a driving axle, must sooner or later give way, unless a lower calculated tensile strain than 5 tons per square inch be put upon the iron.

If the girders are of the lattice type, every care should be taken to provide for the heavy moving loads, by introducing counter bracing, or by so designing the diagonals that they may be capable of sustaining the maximum strains, both of tension and compression, induced by the heaviest engines. In modern bridges designed by our first engineers it is not an unfrequent occurrence to find the calculated compressive strains in the centre diagonals to be about 2 tons per square inch.

Sir William Fairbairn has recorded, in his third series of 'Useful Information for Engineers,' many experiments carried out at the request of the Board of Trade, with the object of ascertaining the effect of rolling loads. Among these, in another form, will be found the following tests:

A wrought iron riveted plate beam, 20 feet clear span, 16 inches deep, nett effective area of bottom flange 1·775 square inches, gross area of top flange 4·3 square inches, web $\frac{1}{8}$ of an inch thick, weight of girder 7 cwt. 3 qrs., calculated breaking weight in the centre 12 tons.

By suitable mechanism put in motion by a water-wheel, loads, the extent of which will be mentioned hereafter, were put upon and taken off this girder at the rate of about eight

applications per minute, continued without intermission day and night.

With a load of 6643 lbs., about one-fourth the ultimate breaking load, the calculated strain in the bottom flange is 6·25 tons per square inch of effective section, the deflection at the centre was 0·17 inch, which was not increased by 596,790 applications of the above load; at this point the load was increased to two-sevenths of the ultimate breaking load, with which the calculated strain in the bottom flange is 7·39 tons per square inch of effective area. The deflection was augmented to 0·22 inch, and after 403,210 applications it was ascertained to be only 0·23 inches. "As the beam had now sustained one million changes of load without any apparent injury," the load was further increased to 10,486 lbs., or two-fifths of the breaking weight, and after 5175 applications the beam broke by tension a short distance from the middle, the calculated strain on the iron at that point being 9·88 tons per square inch of effective area.

The beam, after being repaired, was again subjected to 3,150,000 applications of a load equal to one-fourth of its breaking weight. This load producing no effect whatever, it was increased to one-third of the breaking weight, with which the beam failed by tension after 313,000 applications, the strain in the flange being 8·45 tons per square inch.

The author is of opinion that these experiments fully substantiate his views. If the limit of elasticity of the iron, of which this beam was composed, be assumed to be 8 tons, it will be seen that, in the instance of the 596,790 and 3,124,100 applications, the strain in the flange, being only 6·25 tons per square inch, was well within the assumed limit of elasticity; which, if correct, there is no reason why the girder should not have sustained an unlimited number of applications of the weight equivalent to one-fourth of the breaking weight; this is of course not taking into account

failure occasioned by the giving way of joints or rivets. Notice the difference when two-fifths of the breaking load is applied—only a little over 5000 applications are required to produce fracture; but then the strain on the iron is as much as 9·88 tons to the inch, which considerably exceeds the assumed limit of elasticity of 8 tons per square inch, which cannot be far wide of the mark, as it required over 300,000 applications of a load straining the iron to 8·45 tons to the inch to produce fracture; and with 403,210 applications of a load producing a strain of 7·39 tons per inch, the girder sustained no injury whatever.

Mr. Baker has conclusively proved, in his admirable little work on 'Long Span Railway Bridges,' that there are many circumstances, such as badly maintained permanent way, inclined cylinders, and unbalanced portions of the mechanism of locomotives, together with great weight and length of engines combined with short wheel base, which will at times render the effective load on one axle equivalent to 30 tons. This, taken with the deductions from Sir William Fairbairn's experiments, will almost render it necessary for an engineer to limit the life of railway bridge platforms, constructed as of old with closely spaced shallow cross girders.

When a platform is first constructed, every care is taken to have no two joints in longitudinal timbers on the same cross girder; unfortunately timber is a perishable material, and railway work renewals are often left to the charge of navvies, who, not being well versed in the art of bridge building, sometimes make the longitudinal timbers to joint on the same cross girder, and in the case of a double line of way three out of the four may be found to do so; this is not all the navvie's fault; common sense, of which he has not a little, directs otherwise; but in order that traffic may not be interrupted, a short portion is commenced and finished at a time.

Assuming a single line bridge with cross girders 3′ 6″ apart, originally designed to sustain 15 tons to each axle with a maximum strain on the iron of the bridge not exceeding 5 tons to the inch; reasons have already been given why this load will at times be augmented to 30 tons, the longitudinal timbers be decayed and jointed on the same cross girder; therefore, as the rails will deflect through a considerable portion of the bridge, it will not require more than 5 tons to each axle to produce the amount of deflection in the timber, to correspond with that of the cross girder occasioned by the 25 remaining tons. But since a load of 15 tons to an axle will induce a strain of 5 tons to the inch, that produced by 25 tons will be $x = \frac{25 \times 5}{15} = 8 \cdot 33$; but Fairbairn's beam failed with 313,000 applications of a load producing practically this strain per square inch; therefore, if the bridge be situate on a portion of a line subject to a traffic of sixty trains a day, going full speed, and the excessive load of 30 tons to one axle be only due once to each train, it will require but fourteen years to cause the destruction of the platform. The load of 30 tons may be due to either the leading, driving, or trailing axles; the effect of the engine need only be considered, as the remaining rolling stock will induce strains well within the limit of elasticity. The economy is thus made evident of keeping proper hands to look after a railway, and to see that everything is kept in good condition; bad permanent way not only ruins the rolling stock and passengers' constitutions, but it greatly adds to the wearing out of the bridge platforms. With shallow cross girders, oscillations are set up by heavy continuous traffic, which will soon shake loose rivets, bolts, and perhaps the connections with the main girders.

It may be said by some who have never noticed a worn out bridge platform, that calculations of the foregoing nature

show ignorance of actual work; such assumptions as each train giving one excessive load on a cross girder, may do very well for theory, but prove fallacious when subjected to the crucial test of practice; this may be so, as possibly the excessive load may occur more than once to each train, the constructive weakness referred to doing much to cause this effect; at any rate the author considers that the deductions point out some pitfalls to be avoided, and clearly show the effect of moving loads on girders and analogous constructions. Here is an actual example recorded in the before-mentioned 'Long Span Railway Bridges,' a book which all connected with railway work should read. The platform of the railway bridge over the Regent's Canal was constructed, owing to local circumstances, with cross girders only 8 inches deep and 14′ 6″ span; to compensate as much as possible for want of depth, longitudinal stiffening girders 18 inches deep were placed at a distance of 2′ 3″ from the outer edge of each rail; each cross girder was also well secured by tee irons and gusset plates to the main girders. This bridge, notwithstanding that with 15 tons to one axle it was so designed that the iron should not be strained to more than 4 tons per square inch, completely gave way in four years. Mr. Baker attributes the failure to the employment of a 45-ton engine to work the traffic, the wheel base of which was 14′ 0″; the ends consequently overhung very much, which would greatly assist in producing oscillations and other undesirable consequences.

In good modern practice, closely spaced shallow girders are only used where absolutely unavoidable; it is more often found that the distance between cross girders is 7, 12, or even 20 feet; the latter is an extreme, not recommended on the score of economy; for even admitting, what is not the case, a saving in the weight of platform, still it would probably not be economical to space the triangulations of the

web at 20 feet centres, as the compression members would so be rendered unduly heavy. By introducing another system of zig-zag, the distance between the cross girders is reduced, and no doubt a saving effected in the total weight of the structure.

These and other kindred questions will admit of no general solution, and are therefore quite dependent on the ability of the designer; in warren, trellis, and even plate girder bridges, the spacing of the cross girders is in most cases fixed by considerations affecting the main girders.

So far as approximate weights for taking out strains, and parliamentary estimates, are concerned, the weight of iron in a properly designed double-line bridge platform with two main girders, may be taken at 9 cwt. per foot run, within the limits of 8′ 6″ and 20 feet spacing of cross girders; that is, assuming the iron not to be strained with the heaviest loads to more than 4 tons in the former and $4\frac{1}{4}$ tons per inch in the latter instance; on the completion of the design there will of course be no difficulty in ascertaining the accurate quantities.

	Span.	Total Load on Girders.	Nett Area of Bottom Flange.	Weight of Girders.	Weight per Foot Run of Bridge.
SINGLE LINE.	ft.	tons.	sq. in.	lbs.	lbs.
Cross girders 3 ft. apart	14	17·26	6·30	1206	402
Cross girders 12 ft. apart	14	29·35	10·93	1700	268·2
Longitudinal rail girders	12	19·54	10·80	1518	
DOUBLE LINE.					
Cross girders 3 ft. apart	$25\frac{1}{2}$	35·00	11·40	3654	1218
Cross girders 12 ft. apart	$25\frac{1}{2}$	58·64	19·20	4704	645
Longitudinal rail girders	12	38·64	21·60	3026	

Mr. William Anderson was among the first to appreciate and point out the great economy to be derived from properly spacing cross girders. This is exhibited apart from any con-

siderations affecting the other portions of the bridge, in the preceding Table of estimates prepared by him, and kept on record in the 'Transactions of the Institution of Civil Engineers of Ireland.' Vol. viii. 1866.

It may be remarked that the weight per foot run of platform girders given by the author is considerably in excess of that contained in the table; in the former instance the strain on the iron is taken low, and the damaging action of a 45-ton engine is provided for, whilst in the latter the data assumed are—maximum weight of engine 34 tons, maximum load on driving wheels 16 tons, wheel base 12 feet, depth of cross girders $\frac{1}{12}$th of clear span. The ever increasing complications in running powers, taken in conjunction with the various types of engines used on railways, will necessitate the weight given by the author on almost any railway to be constructed in this country; still the comparisons exhibited in Mr. Anderson's Table hold good, and are of great value. It must not be assumed that in spacing cross girders within the limits of 3 and 12 feet, that the weight of the platform will vary and be proportional to their distance apart.

Discussion.

"Experimental data were given to show that a riveted joint, with the edges of the holes rounded, would sustain a tensile strain very much in excess of that obtained with a similar joint in which the edges were left square."

Author's Reply.—There can be little doubt but that additional strength will accrue from the rounding of the edges of the holes; but the author fears that even were it deemed advisable in practice to institute such a class of work, that the result would fall far short of that above inferred. In

studying fractured axles or other forgings, it will invariably be found that they fail at sharp angles, such as are sometimes formed at a collar or shoulder; therefore, wherever possible, liberal fillets should be employed, in cast, rolled, and other forms of iron. Holes when drilled have generally very much sharper edges than when punched, consequently rivets in properly punched plates bear a greater shearing strain than when in drilled work.

After reviewing the evidence on the matter to be found in the records of experiments made by various eminent men, little doubt can exist as to the damaging effect of the punch when used with common iron; but in dealing with iron capable of withstanding the moderate tests laid down in the paper, little fear need be experienced in embracing the economy to be obtained with this tool; provided there is a full diameter from the edge of the plate to the near side of the hole.

"In punching plates the holes become slightly conical, and it was suggested that the advantages of countersinking may be obtained by placing the plates with the smaller diameter of the holes together."

Author's Reply.—In practice this is sometimes supposed to be done, yet it is seldom aught but a supposition. Except in boiler-work and shipbuilding, rivets are not often strained in the direction of their length, which, so long as the rivet retains its head, is the only case in which the coneing of the holes can be of much service. In these classes of work, and more especially in the former case, holes are generally more or less blind, and therefore are rarely filled without the use of the steel drift; which is very liable to upset the edges of holes placed as advocated, that is, drive the metal between the plates, which is certainly not conducive to a tight joint, the requirement above all others in work of this nature.

The drift cannot be discarded until the better method of

rimering the holes becomes more general; the holes in this case are punched or drilled slightly smaller than ultimately required, and afterwards, when bolted in place, rimered out to the full size, which operation must be executed with care, as the small iron shavings are very liable to get between the plates. By rimering or drifting the coneing of the holes is necessarily done away with; but this is of little consequence, as its whole value depends on the assumption that they are completely filled by the rivets, which, although accomplished in experiments, is seldom found to be the case in practice; if it were, owing to the best workmanship not guaranteeing the holes in the various thicknesses of plates being precisely opposite each other, it would be impossible to get rivets out without the aid of the drill; whereas in ordinary work it is only necessary to knock off the head and out comes the rivet even when through four or five plates. These remarks will not apply to "machine work."

"Machine riveting was strongly recommended by many present, being considered more economical and very much more expeditious than hand riveting, at the same time giving better work."

Author's Reply.—He fully concurs in the favourable opinions expressed in regard to machine riveting, and would advise that it should be employed wherever practicable. A "gang" of riveters, that is, two riveters, a holder up, and one or more nippers, as occasion may require, will, in straightforward work, put in 180 rivets ($\frac{3}{4}$ inch) per day, and when on piece-work they will, without overworking themselves, make time and half, that is, 270 rivets, whilst a machine will easily do this amount of work in $1\frac{1}{2}$ hours, without any additional cost for attendance; that is, not taking into consideration engine drivers, or stokers, as a very few of these will be sufficient to supply the whole works with motive power, a portion of which must be debited to the machine. The advan-

tages of steam riveting are not strictly a modern discovery; a steam riveter was much used in the construction of the Conway tubes, especially where there were many layers of plates; a pressure of 32 tons on each rivet was found to be very much more effective than the blows from 4 lb. or 7 lb. hammers; but, even with this great pressure, it was found necessary to strike the plates round the rivet, and in some cases to leave the pressure on for a short time, in order to prevent the hot iron of the rivet from getting between the plates, which would destroy any advantages to be derived from the friction of the plates one upon another caused by the contraction of the rivet in cooling. Of late hydraulic riveting machines have come much into use; some of these will exert a pressure of no less than 60 tons on one rivet; the pressure of the water in the pipes is maintained at nearly 700 lbs. to the square inch, or about that used in all hydraulic machinery of the class usually employed on dock or other work. In hydraulic riveters this great pressure is economised, by having both a small and large cylinder attached to the levers; the pressure is first put upon the small ram, and the arms worked by it to the required pitch, the final closing of the rivet is then performed by the large ram. A machine of this description giving very good results is patented by Messrs. McKay and McGeorge, an engraving of which will be found in the 'Engineer,' April 24th, 1874.

Diagrams of plates and rivets sectioned by a planing machine, in order to exhibit the qualities of steam and hand, hydraulic and hand riveting, may be found respectively in vol. ii. of the 'Britannia and Conway Tubular Bridges,' by Edwin Clarke, and 'Engineering,' Aug. 14th, 1874.

"It was stated that owing to the contraction of rivets in cooling, the plates will be brought into such close contact,

that the friction between them will be sufficient to withstand the working strain, without any shearing action coming upon the rivets."

Author's Reply.—As this is an important point, a little evidence may be useful. Mr. Edwin Clarke, in his work on the 'Britannia and Conway Tubular Bridges,' makes these observations:

"The contraction of a wrought-iron rod in cooling is about equivalent to $\frac{1}{10000}$th of its length from a decrease of temperature of fifteen degrees Fahrenheit, and the strain thus induced is about 1 ton for every square inch of sectional area in the bar. Thus, if a rivet 1 inch in section were closed at a temperature of 900 degrees, it would, in cooling, decrease in length $\frac{60}{10000}$ths of its length, and if its elasticity and strength remained perfect, would produce a tension of 60 tons. The ultimate strength of rivet iron, however, being only 24 tons, the rivet would in cooling be permanently elongated, and would continue when cool to exert a tension of 24 tons, provided its elasticity remained uninjured by the strain. Thus, if the rivet were not in contact with the plates, excepting at the head and tail, the plates would be held together by a pressure of 24 tons, and this friction would have to be overcome before the rivet came into action as a mere pin."

He (Mr. Clarke) further, with the view of ascertaining how far this theory holds in practice, conducted some experiments from which the following are taken:

Three $\frac{5}{8}$-inch plates were riveted together, with one rivet, in a manner similar to a chain, the hole in the centre plate being oval, and considerably larger than the $\frac{7}{8}$-inch rivet; a weight of 5·59 tons was attached to the centre plate before it slipped, which it did abruptly.

The experiment was repeated with $\frac{1}{2}$-inch washers placed on each side of the outer bars, so making the shank of the rivet

$2\frac{7}{8}$ inches in length; under these circumstances 4·47 tons caused the plate to slide.

The rivet in the last case being assumed to be faulty, the same experiment was repeated, and the plates sustained 7·94 tons before they slipped.

"In the next experiment a $\frac{7}{8}$-inch rivet was inserted through two $\frac{5}{16}$ plates, with large holes, with a $\frac{5}{16}$ washer on each side next the rivet head. This combination supported 4·73 tons before it gave way."

After going fully into the matter, he infers that the Britannia tubes would not deflect more than they do at present, even supposing all the holes to be too large for the rivets. He also points out that rust must be entirely superficial, the close union of the plates preventing any internal oxidation. His remarks are concluded as follows:

"Thus, also, by judicious riveting the friction may in many cases be nearly sufficient to counterbalance the weakening of the plate from the punching of the holes; so that a riveted joint may be nearly equal in strength to the solid plates united."

Mr. Fairbairn, in his 'Useful Information for Engineers,' says:

"From these facts it is evident that the rivets cannot add to the strength of the plate, their object being to keep the two surfaces of the lap in contact, and being headed on both sides, the plates are brought into very close union by the contraction or cooling of the rivets after they are closed. It may be said that the pressure or adhesion of the two surfaces of the plates would add to the strength; but this is not found to be the case to any great extent, as in almost every instance the experiments indicate the resistance to be in the ratio of their sectional area, or nearly so."

More recently Mr. E. J. Reed, late Chief Constructor of the Navy, has had, at H.M. Dockyard, Pembroke, some experiments carried out closely resembling shipbuilding work; they

are here given as recorded in his work entitled 'Shipbuilding in Iron and Steel.'

"In this case three plates were united by what is known as a chain-joint—that is, the ends of the two outer plates overlapped the end of the middle plate. (*Reference was here made to a sketch which has been omitted.*) The connection of the plates was made by three rivets passing through the lap, the rivet-holes in the outer plates being filled by the rivets, but the bearing surface of the holes in the middle plate were slotted out. It will thus be obvious that when a tensile strain was brought upon the middle plate, the amount of the friction could be measured by the force just able to produce a sliding motion. The breadth of the lap was three diameters, the rivets were a diameter clear of the edges of the plates, and the pitch was four diameters. There were two sets of experiments made with iron plates and rivets, and in each set of two, experiments were made with rivets having heads and points snap-headed; two others with rivets having pan-heads and conical points; and the remaining two with rivets having countersunk heads and points. The experiments were made in duplicate, in order to reduce the chance of error. The first set of experiments was made with ½-inch plates, 8¼ inches wide, the rivets being ¾ inch. The results were as follows:

Description of Rivet.	Friction per Rivet.		
	1st Experiment.	2nd Experiment.	Mean.
	tons.	tons.	tons.
Snap-heads and points	5·14	4·21	4·67
Pan-heads and conical points ..	5·26	4·81	5·00
Countersunk heads and points ..	4·56	3·74	4·15
Mean of the three	..	..	4·61

The second set of experiments was made with plates 11 inches wide and ⅞ inch thick, the rivets used being 1 inch

diameter. The following results were obtained under the above-stated conditions of pitch of rivets, lap, &c.:

Description of Rivet.	Friction per Rivet.		
	1st Experiment.	2nd Experiment.	Mean.
	tons.	tons.	tons.
Snap-heads and points	5·84	5·61	5·7
Pan-heads and conical points ..	6·87	7·24	7·0
Countersunk heads and points ..	4·56	4·09	4·3
Mean of the three	..	..	5·6

"In addition to these experiments with iron plates and rivets, two other sets of experiments were made with steel plates and rivets of exactly the same dimensions as those used in the former experiments, the pitch of rivets, breadth of lap, &c., being in each case identical with those previously given. With $\frac{1}{2}$-inch plates and $\frac{3}{4}$-inch rivets, the results obtained were as follows:

Description of Rivet.	Friction per Rivet.		
	1st Experiment.	2nd Experiment.	Mean.
	tons.	tons.	tons.
Snap-heads and points	3·86	4·09	3·98
Pan-heads and conical points ..	4·79	4·79	4·79
Countersunk heads and points ..	3·63	3·43	3·53
Mean of the three	..	..	4·1

"With $\frac{7}{8}$-inch plates and 1-inch rivets, the following results were obtained:

Description of Rivet.	Friction per Rivet.		
	1st Experiment.	2nd Experiment.	Mean.
	tons.	tons.	tons.
Snap-heads and points	6·43	5·49	5·96
Pan-heads and conical points ..	5·49	none made	5·49
Countersunk heads and points ..	5·14	4·91	5·02
Mean of the three	..	..	5·49

"It thus appears that rivets with pan-heads and conical points have the advantage over both the other descriptions of riveting. The only exception to this is found in the second set of the experiments with steel plates and rivets; but as only one experiment was made, the result cannot be relied on. It also becomes evident that counter-sunk riveting causes much less friction than the other systems. On comparison it will be seen that in nearly all cases steel plates and rivets give less friction than iron, the only exception being the cases of rivets with snap heads and points, and those with counter-sunk heads and points, in the second set of experiments. The former of these exceptions is scarcely worth notice, as the difference is so small. The use of larger rivets with the same pitch, &c., gives an increase in the friction, but no law of increase appears to be conformed to."

"Although these experiments do not give any definite idea of the probable amount of friction which would result from the use of rivets having different diameters and pitch, they yet serve to show how much the strength of a riveted joint is increased by the contraction of the rivets."

Mr. Reed, at a more advanced stage of his investigation on this point, says:

"It would consequently be improper to arrange the fastenings of a wrought-iron structure on the assumption that the shearing strengths of the rivets (determined by experiments on rivet bars), and the friction of the surfaces, might be treated as acting conjointly but independently. In the investigations on riveted work which are given in this chapter we shall, therefore, assume that friction is included in the values which are employed for the shearing strength of the rivets."

Mr. Reed considers that his views, as here expressed, agree with those of Mr. Fairbairn, before given. Still the author thinks that there is some disparity in the views held by the

eminent authorities named; and were it not so, he would certainly hesitate to express the opinion that, unless in very superior work, rules in regard to the friction between the plates, deduced from experiments, will, he fears, prove very fallacious in practice; rivets frequently do not only not fill the holes, but they are in addition sometimes positively loose. No doubt loose rivets will be detected by the inspector, and chalked accordingly to be cut out—but are they cut out? If caulking tools and paint could speak, they would have many a tale to tell about loose rivets. Under any circumstances it may safely be assumed that there are few rivets in any work which exert a holding-together strain of 24 tons per square inch; that is the amount deduced by theory, which will be found by calculation to be much exceeded by the experiments just quoted. Even should such be the case, it would not be of long duration, for a very small additional strain will cause iron to considerably elongate, when previously under a strain nearly approaching its breaking point; and the smallest possible additional set will be sufficient to do away with any advantages to be obtained by the contraction of a rivet in cooling. It is not here the intention to imply that the frictional resistance of the plates will not add a certain extent of strength to a joint, but the author is of opinion that with vibration any additional strength caused by this resistance will be destroyed; and that, therefore, the old rule, of making the shearing area of the rivets on one side of the butt equal to the effective area of the plates in tension, should be adhered to; and as the shearing strength of the rivet is not equal to the bar from which it was made, a few over and above the number thus deduced will in the author's opinion neither do any harm nor add greatly to the cost of the structure.

"The mean tensile strain of 13·07 tons per square inch, obtained in testing the cast iron of which the sleepers of the

Great Indian Peninsula Railway sleepers were made, was thought to be an extreme not often reached."

Author's Reply.—Undoubtedly this is a higher strain than most cast iron used in construction in this country will be found to stand; still, by proper treatment, care, and an expenditure of money, it may be much exceeded. The careful experiments carried out by the United States Government, and recorded by Major Wade, of their Ordnance Board, in a volume entitled 'Reports of Experiments on the Strength and other Properties of Metal for Cannon, &c.,' give the maximum tensile strain obtained with cannon iron at 20·5 tons per square inch; and the mean of samples cut from one hundred guns was 14·9 tons. These high results are obtained by remelting the iron, sometimes three or four times, and keeping it at each melting in a state of fusion for from one to four hours. As an instance of what may be effected in this way, samples cut from twenty-seven guns sustained a tensile strain of 15·75 tons per square inch; whilst the crude pig iron from which these guns were cast only stood 5·66 tons per square inch. More detailed information concerning these experiments may be found in a paper on "American Iron Bridges" read by Mr. Zerah Colburn at the Institution of Civil Engineers, May 5, 1863. In the discussion which ensued on this paper, Mr. Bramwell gave some interesting particulars of similar experiments.

"He exhibited two samples of Acadian cold blast iron, the produce of Nova Scotia, which had been broken in his presence by the proving machine at Woolwich Arsenal. Being No. 1 iron, a mixture of scrap was required before strength could be obtained. But as the experiments were made to test the quality of this individual iron, it would have rendered those experiments nugatory had scrap from any other source been mixed with the iron. The only way of proceeding that occurred to him was to melt some of the

iron into pigs to make scrap for the second melting, reserving trial bars of the first melting. The order of the experiments was as follows: The iron was put into an air furnace, and as soon as it was fused eight sample bars were cast. These bore an average tensile strain of $7\frac{1}{2}$ tons per square inch of section. The iron was kept in fusion, and two hours after, eight other bars were cast. These did not break until a strain of 8 tons 6 cwt. was attained, being nearly 1 ton more per square inch than the strain which broke the first lot of bars. After a further interval of one hour and three-quarters, eight other bars were cast, which broke with a strain of 10·8 tons; so that by keeping the metal in fusion for three hours and three-quarters the tensile strength was increased from 7 tons 10 cwt. to 10·8 tons, or about 50 per cent. The iron was then poured out, and pigs were cast from it. On the next occasion the furnace was charged with half of the fresh No. 1 iron, and half of the pig iron from the prolonged fusion. The result was that the bars cast immediately upon melting required for their fracture a tensile strain of 11 tons."

Bars which were cast after four hours fusion stood an average tensile strain of 18·5 tons, and the maximum of these samples stood as much as 19·6 tons.

RETAINING WALLS.

Being a Paper read at a Meeting of the Edinburgh and Leith Engineers' Society, May 6, 1874; PROFESSOR ALEXANDER B. W. KENNEDY, *Past President, in the Chair.*

To a great extent the sections of retaining walls are deduced from those already constructed, a mode of proceeding fully justified by the many complicated conditions appertaining to their design.

On them and a few other works in Civil and Mechanical Engineering, is founded the "miscalled practical man's" contempt for the theory by which he, unknowingly, is enabled to carry out any work.

The engineer and really practical man is neither slave to precedent nor the dictates of theory, but each is made subservient by turns to the other, both being so blended as at last to obtain that harmonious whole, which remains a tribute to thought, perseverance, and skill.

Whatever the shortcomings of theory based on the sound laws of mechanics, and worked out by those of mathematics, the verdict of the most unfavourable jury can only be, that the theoretical structure is one of uniform strength; and would be so in practice, if the premises were obtainable on which the theory is based; unfortunately, this is not so, and thus the abstruse reasoning of our Continental brethren is rendered of little use; more especially in ironwork, where the strains in tension and compression bars are often varied

more than 25 per cent. by nuts, cotters, and riveted joints being improperly made.

These contingencies, with 10 per cent. added for the elasticity of the material, induce engineers in this country to discard, where optional, abstruse calculations and complex formulæ, in favour of the graphical methods of representing strains.

What can be more simple or practically more accurate than diagrams representing the moments, rivets, plates, and sectional areas of the iron in a girder, by which can be ascertained at a glance the positions of joints in plates, as well as the economical and general distribution of the metal?

For although mathematics may be an easy mode of reasoning, still when the reason is guided at each step by the eye and a knowledge of general principles, the conclusions arrived at will in all probability be more sound than those obtained by dealing with x and y, the values of which are unknown until the calculation is completed.

From the foregoing it will be seen that in subsequent investigations, the author does not pretend to greater nicety than is consistent with our knowledge of materials and the practical carrying out of work.

In endeavouring to proceed in a concise manner, it has sometimes been necessary to employ slightly dictatorial language and to express unqualified opinions, in the reception of which the meeting must make every allowance, as it has only been done, where demanded, to save words.

With masonry the 10 per cent. of contingencies caused by the elasticity of materials may be dispensed with, as also in good ashlar a considerable portion of that for workmanship, the properties of good ashlar being as well or better known than those of ironwork; and so it is the strains in stone columns and arches may be determined with more ease and accuracy than in those of iron.

A wall when properly built should be as nearly as possible

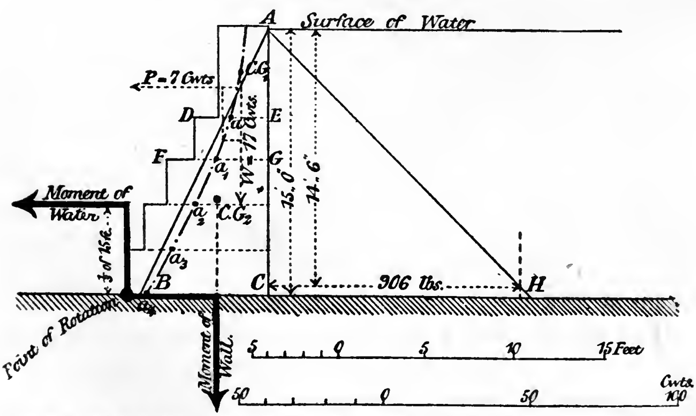
equal in strength and stability at all points; any portion, therefore, being cut off by a level line from the bottom, must simply be assumed as resting on the under portion as a foundation, and the adhesive force exerted by the mortar not considered, as this class of work is seldom constructed with the care necessary for structures in the design of which the tensile strength of the mortar enters to any great extent.

Theory and practice being closely allied in the construction of walls to support still water, they will be first taken as being the most simple, and as gradually leading up to the more complicated problems involved in the investigation of walls to support earthwork.

In dealing with fluid pressure, there is for a base of operations the well-known hydrostatical law: The pressure of a

Fig. 1.

Diagram illustrating the Theory on which the Stability of a Retaining Wall depends.



CG_1 is the centre of gravity of section above line DE, CG_2 of whole wall.
"Line of resistance" of wall shown thus, ——•——•——•—— a a_1 a_2,

fluid on any plane is equal to the weight of a prism of the fluid, whose base is equal in area to the plane, and height to

the distance from the surface of the fluid to the centre of gravity of the plane; and in any rectangular plane which has one edge in the surface of the fluid, the pressure acts in a direction perpendicular to the face of the plane, at a point termed the "centre of pressure," situate at a distance of two thirds the depth of its immersion.

As examples generally convey a meaning more clearly than written explanations, they will be resorted to, and Figs. 1 and 2 have been prepared for reference.

Rubble masonry is taken as weighing 130, brickwork 112, and water 62·5 lbs. to the cubic foot; the length of wall under consideration is always one running foot, and a cubic foot will in all cases be the unit of measure. *The toe of the wall,* as hereinafter employed, is the point where the surface of the ground cuts the foot of the wall.

Salt water being more dense than fresh, weighs heavier in proportion to the amount of salt in solution; but if 3 per cent. be added to all calculations with fresh water, sufficiently accurate results will be obtained.

The pressure against the back of a wall is nothing at the surface, and increases per foot of area as we descend each foot, only by the weight of an additional foot of water, until at the base of the wall it will be that due to a column of water one foot in area; and in the present instance 14′ 6″ in height, weighing 906 lbs. Therefore, if a line be drawn from the point C, Fig. 1, at right-angles to the back of the wall, equal in length to its depth below the surface, and the length of this line represent the pressure at the bottom of the wall in lbs.; another line being drawn from its unconnected end H to the point A where the surface of the water cuts the wall, will enclose a triangle A H C, the area of which will represent the total pressure against the wall, and its ordinates the pressure in lbs. per foot of area at any point.

As the weight of water at the surface is nothing, here

theoretically the thickness of the wall will also be nothing, and the pressure of water per foot of area increasing uniformly as the ordinates of a triangle, it follows that the wall to offer a uniform resistance will also have a triangular section, and the areas of these triangles will be inversely proportional to the weights of masonry and water; or, in figures, the base B C of the triangle A B C : 15 :: 62·5 : 130, therefore the base B C=7·2 feet. The triangle A B C is then the theoretical form which must be adhered to as nearly as possible in practice, although, in order to be on the safe side, employing somewhat larger dimensions; which with this as a starting-point can be readily deduced by the Practical Engineers' Calculus, or the most useful but rather unscientific method of trial and error.

Practice will determine the minimum thickness of the wall at the top, which must also be made to conform to theory, and be modified to obtain an economical section of wall. In this case assume a portion of the wall above the line D E to be of rectangular section, and 3′ 0″ in thickness; the water acting against this is 5′ 0″ in depth, and will exert a thrust against the wall of

$$\frac{5 \times 2{\cdot}5 \times 62{\cdot}5}{112} = 7 \text{ cwts.};$$

the face of the wall being vertical, this will act horizontally at a point two thirds of 5, or 3′ 4″ from the surface; this force must be transmitted to the foundations through the mass of the wall; and assuming the foundation to be unyielding, it must be resisted by its weight, which acts vertically downwards through C G_1, its centre of gravity, and so far as the overturning of the wall is concerned, there will be two forces acting at right angles to one another, represented on Fig. 1 by P and W.

Now by mechanics. If two forces acting on a particle be represented in magnitude and direction by two adjacent sides

of a parallelogram, the resultant of these forces will be represented in magnitude and direction by that diagonal which passes through the particle; and also, on the same authority, if the resultant pass without the base of the wall it will be overturned: in the case under consideration it does not do so. Again, by ascertaining the weight, first of the water acting on the section of the wall above the line F G; and secondly, of the wall, which will act vertically downwards through the centre of gravity of the whole section above F G, and proceeding in a manner similar to that already described, the point a_1 will be found, and so on any number of other points a_2, a_3, a_4, by joining which will be obtained roughly what is termed the *line of resistance* of the wall.

Owing to the liability of masonry to fail if subjected to too great a crushing strain, and to ensure sufficient stability, the line of resistance must be some distance within the body of the wall, by some authorities even as much as one third of the thickness; but from a study of existing structures, the author is led to believe, that if the distance between it and the nearest face be not less than one fifth of the thickness of the wall at that point, then the condition of safety will be amply fulfilled.

Keeping strictly these conditions in mind, little difficulty will be experienced in so modifying the section that it may be made to comply with this, or other requirement; and should any doubt exist as to its stability after being so changed, the investigation on the altered section may be repeated in a few minutes, or a check may be put upon it by the system of moments, which it will, perhaps, be well here to describe.

A retaining wall will be stable, if the moment of its weight round the point of rotation exceed, by a certain factor of safety, that of the thrust of the water or bank round the same point.

In the present instance the weight of the wall is 88, and the thrust of the water 63 cwts., and by scaling the lengths of the arms at which these forces act from Fig. 1, we find the moment of the water to be

$$63 \times 5 = 315 \text{ cwts.,}$$

and that of the wall

$$88 \times 5 = 440 \text{ cwts.,}$$

or a little more than one third greater than the moment of the water; which the author considers a sufficiently large factor of safety, as that deduced from the condition of the line of resistance passing $\frac{1}{5}$th within the thickness of the wall, requires only an excess of $\frac{3}{10}$ths, or a little less than one third; this discrepancy in the results is owing to the diagram of the wall not being constructed with absolute theoretical exactness.

Were this wall designed to support water, it would be advisable to give it a still greater factor of safety; but such is not the case, water only having been chosen as the means of illustrating the theory on which the stability of a wall depends.

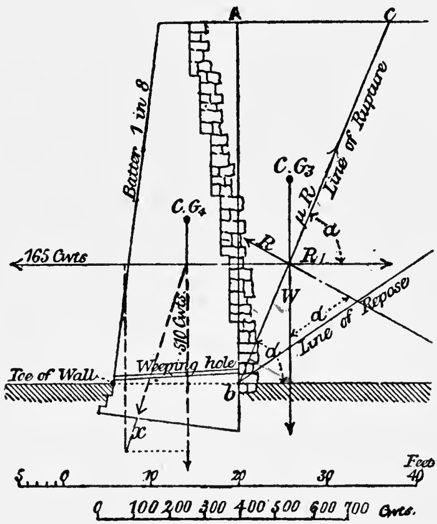
If the wall is of good rubble masonry, and is required to support a head of still water not exceeding 25 feet, the author has found by calculation that the most economical batter to the inner face will be 1 in 4, being that to which the reservoir walls of the Dunkerque Waterworks are built; but for large masonry dams it is admitted, by English as well as Continental engineers, that the inner face should be nearly plumb, having a curved batter of something like 1 in 15. Apart from stability, walls or dams to support water tax ingenuity to no little extent, in order to render them water-tight; the embankment with masonry faces, and a puddle-wall varying from 1′ 6″ to 3′ 6″ in thickness in the centre, being that which meets with most favour.

In dealing with earthwork, the precise manner in which it acts against the wall is to some extent indeterminable, owing to the cohesion and friction of its particles; but by experiment and research these have been ascertained with sufficient accuracy for practical purposes.

The amount of earth pressing on a wall is that portion which would immediately fall on its removal, it has been found that this is triangular in shape, and slips on a line, termed the *line of rupture*, which bisects the angle made by a vertical and the natural slope of the material; the centre

FIG. 2.

DIAGRAM ILLUSTRATING THE THEORY ON WHICH A WALL TO SUPPORT EARTH PRESSURE IS DESIGNED.



Section of wall required, if thoroughly well backed up and efficient weeping holes provided, and no weight placed upon the bank.

C G_3, centre of gravity of prism of earth; C G_4, of wall, including earth and dry stone to the left of line A *b*.

of pressure is as with water at one third of the vertical height from the toe of the wall. The prism of earth acts as a wedge,

the back of the wall and the line of rupture being the reacting surfaces; the friction between the back of the wall and the earth is very small in a wall with a plumb back, and for lack of sufficient data is generally disregarded; but that of the earth upon itself is no inconsiderable amount, for although the line of rupture approaches nearer to the vertical than the natural slope, still the wall keeps the wedge in place, and, consequently, the frictional resistance to sliding along the line of rupture will equal in amount the normal resistance of that plane multiplied by a coefficient of friction. Fig. 2 represents the system of forces which must be in equilibrium, in order that the wall may be stable; W is the weight of the earth acting vertically downwards through C G_3, the centre of gravity of the triangle A *b c*; R the resistance of the bank acting normal to its surface; μR the resistance to sliding along the line of rupture; and R_1 the resistance of the wall. If these forces be resolved along, and at right angles to the line of rupture, the following equations are obtained:

$$\mu R + R_1 \cos. a = W \sin. a$$
$$R = R_1 \sin. a + W \cos. a$$

Transposing the first of these

$$\mu R = W \sin. a - R_1 \cos. a$$

Multiplying the second by μ

$$\mu R = \mu R_1 \sin. a + \mu W \cos. a$$

Therefore,

$$\mu R_1 \sin. a + \mu W \cos. a = W \sin. a - R_1 \cos. a$$
$$\mu R_1 \sin. a + R_1 \cos. a = W \sin. a - \mu W \cos. a$$
$$R_1 (\mu \sin. a + \cos. a) = W (\sin. a - \mu \cos. a)$$
$$R_1 = W \frac{\sin. a - \mu \cos. a}{\mu \sin. a + \cos. a} \text{ (Equation I.)}$$

From a careful study of the able investigations of Morin, Roudelet, and others, the author concludes that, so far as the object of this paper is concerned, the slopes formed by various materials, if left to themselves, may be assumed to make with the horizontal the angles tabulated in column number 1, in the following Table, termed in engineering literature the *angle of repose.*

Name of Material.	1. Angle of Repose.	2. Coefficient of Friction.	3. Weight per Cubic Foot.	4. Angle of Rupture.
			lbs.	
Sand and loose earth	32°	·62	100	61°
Shingle and gravel	40	·84	105	65
Compact earth	50	1·19	110	70
Well-drained clay	45	1·00	120	67½
Wet clay	17	·31	120	53½

The coefficients of friction are the natural tangents to the tabulated angles of repose; the weights are the averages of several authorities modified to a certain extent by the author's experience; and the angles made by the line of rupture and the horizontal are obtained by dividing the complement of the angle of repose by two, and adding the result to the angle of repose; or it may be found by adding half the angle of repose to 45°.

The values given for wet clay may be disregarded, as it is seldom, and ought never to be, filled in behind walls; under any circumstances, weeping holes, drains, and other means are provided to carry off the surface drainage and backwater.

It is usual to specify no tipping to be allowed behind the walls, and the filling to be of such material from the excavations as may be directed, placed in layers, inclined slightly upwards towards the wall, and well punned—the last a requirement more often specified than carried out.

If the wall is for a purpose such as a dock, the earth is usually benched, so forming a series of steps on which the ends of the layers of earth rest. Taking these facts into consideration, the author can see no reason why constant values cannot be given to the unknown quantities in Equation I. As it is impossible to compress earth into the hole from which it was taken, filling will approach, but fall short of, compact earth; the angle of rupture, when dealing with other material than sand or loose earth, may be taken at 65°, which is the same as that given in the Table for shingle and gravel, and the coefficient of friction μ will certainly not be excessive if taken at ·84.

By substituting these values in Equation I.

$$R_1 = W \frac{\sin. 65^\circ - \cdot 84 \cos. 65^\circ}{\cdot 84 \sin. 65^\circ + \cos. 65^\circ}$$

$$= W \frac{\cdot 906 - \cdot 84 \times \cdot 423}{\cdot 84 \times \cdot 906 + \cdot 423}$$

$$R_1 = \cdot 47 \text{ W (Equation II.)}$$

and since W represents the weight of the right-angled triangle A b c, Fig. 2, the area of which is equal to $h^2 \dfrac{\tan. 90 - a}{2}$ and the weight $= w_1 h^2 \dfrac{\tan. 90 - a}{2}$ where w_1 equals the weight per cubic foot of the material, assumed in the following calculations at 105 lbs.; and h the height in feet from the toe to the top of the wall; therefore $R_1 = w_1 h^2 \dfrac{\tan. 25 \times \cdot 47}{2}$, which when further reduced becomes

$$R_1 \text{ in lbs.} = 11 \cdot 5\, h^2. \quad \text{(Equation III.)}$$

In the case of a wall 40 feet in height this will give a thrust of $40 \times 40 \times 11 \cdot 5 = 18{,}400$ lbs. or 165 cwts. As before stated, this formula is only applicable to walls properly

backed up, and when due provision is made for drainage. A very good and much used expedient to ensure the filling behind a wall being tolerably free from water, is to build dry rubble, about one foot in thickness, at the back of the wall, previous to filling in; and when constructing the walls weeping holes should be provided about every ten or twelve feet along the bottom of the wall, just above the surface of the ground or the drains into which they are to discharge; the dry rubble must only be carried a few inches below their level; it is as well also to have weeping holes intermediate between them, situate about half the way up the wall; but in work with any architectural pretensions they must be dispensed with.

By calculating with either Equation II. or III. the thrust at one or more points, and proceeding by the trial-and-error system already described, little difficulty will be experienced in arriving at a section, such as that shown in Fig. 2, where the line of resistance cuts the base of the wall at the point x about one fifth within its width. The earth and dry stone resting directly above the steps at the back of the wall have been assumed as resting on the wall, and been taken as part and parcel of the wall, the weight having been considered the same as the masonry; for although it is actually less, still if the foundation of the wall does not fail, the earth must be sheared at least along the line $a\ b$ before the wall can give way; the extra weight has been allowed to compensate to some extent for the cohesive and frictional resistance of the earth, the whole weight of the wall and earth, to the left of the line A b, will act through C G_4, the centre of gravity of the compound mass. By treating the matter in this way, and considering the thrust as always horizontal, the abstruse calculations involved, if the back is shown battered on the drawings, are eliminated. The earth to the left of the line A b is the only portion producing these complications, and by

assuming it as part of the wall, it will be found that the line of resistance of the wall will take practically the same position as that found by treating the thrust as normal to the batter. Even were it not so, walls designed with a straight batter from top to bottom at the back, are built in a series of steps, varying in size according to the dimensions of the stones at hand, so that the horizontal thrust as herein described will hold under all circumstances.

This advantage of placing steps behind a wall cannot be obtained without a waste of material if too great a batter be given to the face. In practice, a batter of 1 in 8 has been found to give economical results.

Stepping the back of a wall was much in favour with Brunel, who often built a thin face wall of nearly uniform thickness, having a batter of 1 in 5, with counterforts at the back 9 or 10 feet apart, the face wall having projecting horizontal steps running the whole length of the wall at the back, and spaced vertically 7 or 8 feet apart; these must have gripped the earth and prevented the wall from overturning, for those walls of the above description which have come under the author's notice would be inadequate of their own weight to do the duty imposed upon them.

It may be remarked that the wall, Fig. 2, is somewhat larger than that in many cases employed on railways. In the case of a wing wall, it is usually well bonded into an immovable abutment at one end, and at the other is made, owing to the exigencies of construction and for appearance sake, very much larger than theory requires; so that the portions of little stability are supported by those of great stability; and the wall is often built to a curve on plan, the advantage of which is sometimes over estimated, as it can be of little use, unless the centre ordinate of the curve is greater than one sixteenth of the chord.

If the wall is backed up with improper material, or has to

support sand or loose earth, by referring to the Table, and taking $a = 61°$, $\mu = \cdot 62$, $w_1 = 100$ lbs., and proceeding as before, it will be found that

$$R_1 = \cdot 56 \text{ W} \qquad \text{(Equation IV.)}$$

$$R_1 \text{ in lbs.} = 15 \cdot 5\, h^2 \qquad \text{(Equation V.)}$$

which for a bank 40 feet in height will give a thrust of 24,800 lbs., about one third greater than that before deduced for a wall properly backed up.

In dock and river walls much larger sections than even that deduced from the last-named example are employed, as at one time they are left high and dry, and at another almost covered with water, which percolates through them and thoroughly saturates the earth, it may be causing it to exert a greater thrust than that resulting from the same head of water; in fact, it would be almost impracticable to provide a wall of sufficient section to support thoroughly wet clay; great economy must therefore result from the practice of making walls watertight, as is to be done at the new Dock Works at Leith, by placing a puddle wall some 2′ 0″ in thickness at the back, it is then a comparatively simple matter to provide a wall to resist the thrust of the bank and that occasioned by the maximum amount of loading allowed on the quay.

In the case of walls supporting banks on which railways, houses, or structures of any kind are to be constructed, it will practically only be necessary to ascertain the total effective weight of bank and structure, and insert it for W in Equation II. or IV. as the case may be, and the resulting thrust will be lbs. or tons, according to whether W has been taken in lbs. or tons.

Should the meeting not approve of the author reducing walls to two standards, values may be obtained from the Table and substituted to suit any individual opinion.

Of course, if the foundations of a structure are carried

below the line of repose of the materials, little or no additional thrust on the wall will be occasioned; and if the weight of the building is made properly to rest on a wall, its section may in many cases be reduced, providing the area of the base is made amply sufficient to form a proper foundation for the superincumbent structure, and too great a crushing strain be not put upon the material. The fact of additional weight on the top of a wall adding to its stability was well understood by the ancients, as may be seen from Gothic architecture, where the weight of the ornamental pinnacle to a flying buttress is often that necessary to render it sufficiently stable.

Other instances of this nature suggest themselves, but it is impossible to reduce the varied problems met with in engineering to any rule of thumb, only the educated engineer can meet successfully unprecedented works, such as those involved in the construction of the Metropolitan Railway.

Many have written on retaining walls, but when the point is reached at which surcharged walls ought to be discussed, little or worse than nothing is said, the reader being made to wade through abstruse equations, generally only to lose himself in pages of coefficients; and it is sometimes stated that the "centre of pressure" is situate at one third the height of the wall from the toe. As it is impossible that this can be the case, the author has appended a clear and simple proof, by Mr. Hurtzig, Stud. Inst. C.E., showing that the centre of pressure will be at one third of the height from the foot of the wall to the point where the line of rupture cuts the surface of the ground.

The slope of the surcharge will often vary according to the opinion of the engineer, no general formula can therefore be given, but if the surface be plotted, and the line of rupture be drawn, the height can be measured, and the weight to be supported ascertained.

The line of rupture is constant for the same material, whether the wall be surcharged or not; at least, if this be

assumed, any error will err on the safe side. Therefore if the material to be supported be compact gravel, shingle, or firm earth, and proper precautions be taken to ensure efficient drainage, the line of rupture may be taken as before at 65°, and the thrust ascertained by multiplying the area of the triangle; of which the sides are, a vertical drawn from the foot of the wall at the back, the slope of rupture, and the surface line of the ground; by the weight per cubic foot of the material, and placing this value for W in Equation II. If the bank to be supported is of loose earth, sand, or clay, the angle of rupture may be taken at 61°, and the value of W under these altered conditions must be placed in Equation IV., and the centre of pressure scaled from the plan as before described.

In the case of surcharged walls, results will often be modified by extraneous circumstances; if the precaution be taken to shore up the earth as the foundations of the wall are taken out, and the earth be so prevented from moving, the wall when constructed will have to resist a very much less pressure than if the earth be allowed to move in the very smallest degree. The cutting in the bank should be protected as soon as it is practicable; for clay, a most treacherous material (as regards walls), when first opened will stand with nearly a perpendicular face, and remain so unless attacked by weather or water; the bank should therefore not be touched until it is decided to proceed with the construction of the wall.

Footings are generally given to the toe of a wall, as although the tensile strength of the mortar does not enter into calculations, still it does exert some influence on its stability; and therefore to make the wall of equal stability at all points, it must, when founded on earth, have one or more footing courses, as there will certainly not be the same adhesion between the earth and the masonry as in the masonry itself.

They are also necessary to distribute the pressure over the, to certain extent, yielding foundation. But if a wall is properly designed and built on rock or concrete, and the mortar well spread over the foundation, the footings can be of little use, as they only make the wall more stable at that point than at any other: they may be required to give additional strength to the concrete projecting beyond the base of the wall, but where it is 2′ 6″ or 3′ 0″ in thickness, and projects only 1′ 6″ or 2′ 0″, no fear need be entertained of its failure; that is, if it is concrete properly made with hydraulic lime or Portland cement, and not a mechanical mixture of stones and mud, with rich lime, as may be sometimes seen in a walk through London.

Rich lime should never be used for concrete or thick masonry, unless the same is protected from all weather and moisture, and even then it will give but very poor results; for, where used in large masses of masonry, it has been found to be perfectly soft in the interior after a lapse of twenty or more years.

If the conditions of stability be complied with, it will in most cases be found in a properly constructed wall that the frictional stability at any point will also be ample; that is, the horizontal thrust at any point must not exceed the weight of the masonry above that point multiplied by the coefficient of friction, which for masonry and brickwork may be taken at ·6. This is a low value, for rubble, when well constructed, becomes one solid mass; as exemplified by the rubble walls on the Mersey Dock Estate, where in some cases the whole body and sometimes the face is composed of random rubble, the pieces of stone averaging not more than 30 or 40 square inches on the face; but, as before stated, rubble is not often constructed with care or reaches this standard.

The condition of frictional stability may be otherwise stated thus: in order that the joints may not slide on one

another, a tangent to the line of resistance at any point must not make a less angle with the bed-joint than the complement of the angle of repose of the masonry or brickwork, which is about 31°.

The tangent to the line of resistance of the wall must therefore not make a less angle with the bed-joints than 59°, consequently the more the joints are inclined the greater will be the resistance of the stones to sliding on one another; but this must not be carried too far, for if the incline is greater than 31°, the portions of the wall will tend to slide towards the bank; but this angle represents an incline greater than 1 in 2; therefore, although there seems to be a conventional rule in the profession that the bed-joints shall be normal to the face, the author can see no reason why all bed-joints in retaining walls with either plumb or battered faces should not be built at an incline of from 1 in 6 to 1 in 8.

Again, the angle of repose of masonry on clay varies between 18° and 27°—it may be assumed for ordinary purposes to be 20°—the tangent to the line of resistance at the point where it cuts the earth foundation must not therefore make a less angle than 70°; as before, the more the foundation is inclined the greater will be the resistance to sliding, and as the angle of repose represents an incline greater than 1 in 3, there can be no reason why the foundation should not also always be made at an incline of, from 1 in 6 to 1 in 8.

The masonry beds might be inclined even more than 1 in 6, but no advantage would accrue, as the frictional stability of the masonry would only be rendered greater than that of the foundation.

So far the foundation has been considered immovable, and the wall to revolve round its toe as a fixed point; in order that theory may hold good, all practicable means must be employed to fix the toe, and render the foundation as

nearly as possible unyielding: to effect which, piles may be driven through the loose earth to firm strata; or if these cannot be reached, the foundation must be spread out by concrete, or counterforts may be used; but, unless the foundation is bad, they should only be used at great distances apart, to give sufficient stiffness to the wall, as, theoretically, walls with counterforts are slightly, and when constructed of masonry practically not at all, more economical than walls of uniform section; but with brickwork they may effect a slight saving. It is sometimes urged, as a reason for their adoption, that walls of thin masonry being more open to inspection, better masonry is secured. This is open to question, for if those employed to construct the work wish to scamp it, they will carry out their object in the face of the most diligent inspection, whether the wall be thick or thin. Walls of masonry or brickwork combined with concrete will in most cases effect a saving of 20 per cent.

If the foundation, or the material to be supported, be very bad, it will, perhaps, be better to employ a thin-face wall and very long counterforts, with inverts and one or more tiers of relieving arches between them. The stability of such a wall is subject to the previously-mentioned conditions, providing the arches and counterforts are of sufficient strength for the work imposed upon them; the weight of this wall, to be used in calculations, will equal that of the masonry or brickwork together with the earth resting directly on the arches.

In conclusion, it is needless to state that the geometrical constructions employed are subject to the most minute mathematical investigation, and, if preferred, it may with little difficulty be substituted.

2, Queen Square Place, Westminster, S.W.,
April 24, 1874.

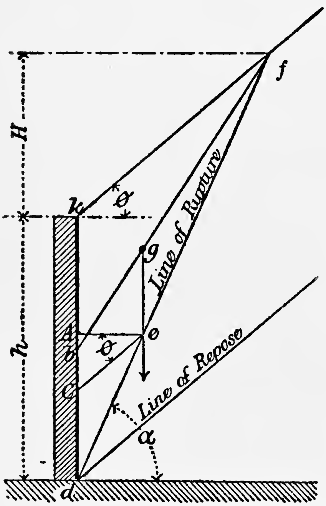
ADDENDA.

Surcharged Walls.

Position of centre of pressure proved by Mr. Hurtzig, Stud. Inst. C.E.

Referring to Fig. 3: Let h equal height of wall, H that of surcharge, a equal angle made by line of rupture with horizontal, and θ equal angle made by surcharge with the horizontal.

FIG. 3.



Bisect $d\,k$ in b and join $b\,f$. On line $b\,f$ take $b\,g$, equal to $\frac{1}{3}\,b\,f$, then g is the centre of gravity of the prism of earth tending to overturn the wall.

From g draw vertically $g\,e$, cutting the line of rupture in the point e, and through e draw e A, horizontally, cutting $d\,k$ in A, which is then (Mechanics) the centre of pressure; through e draw $e\,c$ parallel $k\,f$.

Then because the triangles $d\,c\,e$, $d\,k\,f$ are similar (Euclid VI., 4),

$$d\,c : d\,k :: d\,e : d\,f.$$

And, again, because the triangles $d\,f\,b$ and $e\,f\,g$ are similar,

$$b\,g : b\,f :: d\,e : d\,f;$$

therefore $$d\,c : d\,k :: b\,g : b\,f.$$

But $$b\,g = \tfrac{1}{3}\,b\,f; \text{ therefore } d\,c = \tfrac{1}{3}\,d\,k = \frac{h}{3}.$$

Now $A\,e$ being horizontal, and $c\,e$ parallel to $k\,f$, the angle $A\,e\,c$ is equal to θ, and by trigonometry $A\,c = c\,e \sin.\,\theta$.

From the similar triangles $d\,c\,e$, $d\,k\,f$,

$$d\,c : d\,k :: c\,e : k\,f;$$

and because $$d\,c = \frac{d\,k}{3},$$

therefore $$c\,e = \frac{k\,f}{3}.$$

Now $$A\,c = c\,e \sin.\,\theta;$$

therefore $$A\,c = \frac{k\,f}{3} \sin.\,\theta.$$

By trigonometry $$k\,f \sin.\,\theta = H;$$

therefore $$A\,c = \frac{H}{3}.$$

And since $d\,c$ is equal to $\frac{h}{3}$, and the height required

$$d\,A = d\,c + A\,c,$$

therefore $$d\,A = \tfrac{1}{3}(h + H).$$

That is, one third of the whole height from the toe of the wall to the point where the line of rupture cuts the surface of the bank.

Discussion.

"Rats were by one Member considered to be the natural enemy to all kinds of embankments; he stated that they could in a short time make their way through a puddle wall, so causing in many cases a leak which might lead to very disastrous consequences."

"It was questioned if the properties of good ashlar are as well or better known than those of ironwork."

Author's Reply.—Engineers frequently differ more than 10 per cent. in estimating the rolling loads of bridges and wind pressures for roofs, whilst in selecting materials from those at hand for backing up a wall, the choice of half a dozen engineers will in all probability be unanimous; the material being selected, its weight and other qualities can by experience or experiment be ascertained almost within the above limit.

In ironwork heavy rolling loads may nearly double or totally reverse the strain in some members, so that those which were intended to resist tension may be subject to compression, and *vice versâ.*

Nuts and cotters are frequently used in this country in roofs, and still more frequently in America for bridge construction; the strains in structures with these fastenings are to some extent dependent on the strength of the man, and the length of the spanner with which the nuts are tightened up, or the weight of the mallet employed to drive home the cotters. This is not a random statement, as will be found by calculation. If one turn of a screw-thread be unwound, it will form an inclined plane of which the circumference of the screw will be the base, and the pitch the height; so that the screw simply becomes an application of the inclined plane, in which the plane is moved by a horizontal force, therefore the following relations exist: P the power is to W the resistance or weight to be lifted, as the height of the plane is to its base, W will therefore equal $\frac{P\,C}{p}$, where C equals the base and p the pitch of the screw. This assumes the power P to be applied at the circumference of the screw, which is rarely the case, the screw, as in the present instance, being generally combined with a lever, the power applied at the end of which

must be reduced to its equivalent at the circumference of the screw; not at all a difficult matter, if F be the power applied at the end of the lever of length L, and r equal the radius of the screw; then, neglecting friction, in order that the system of forces may be in equilibrium, or in a state bordering on motion, $Pr = F\,L$, therefore, $P = \frac{F\,L}{r}$. Substituting this value of P in the equation $W = \frac{P\,C}{p}$ the expression obtained is $W = \frac{F\,L\,C}{rp}$; but C being the circumference of the screw, is equal to $2\,\pi\,r$, therefore $W = \frac{F\,2\,\pi\,L}{p}$, but $2\,\pi\,L$ is the circumference of the circle described by the power F. Therefore, neglecting friction, W the resistance or weight to be lifted is to F the power at the end of the lever, as the circumference described by F is to the pitch of the screw.

Applying this to an ordinary Whitworth screw, $1\frac{1}{2}$ inch diameter, and six threads to an inch, the nut of which is screwed up with a key or spanner 2′ 0″ long; as a man may with ease exert a pull of half a cwt. for a short space of time, W is to half a cwt. as 151 inches, the circumference of a circle having a radius of 2′ 0″, is to $\frac{1}{6}$th of an inch, that is the pitch of the screw. W therefore equals 453 cwts., or a little over $22\frac{1}{2}$ tons. The length of the spanner may further be increased to 5 feet, by means of a crowbar with clips provided for the purpose. Under these altered conditions, and neglecting friction, a man capable of exerting a power equal to half a cwt. for a short space of time will be able to put a strain of $56\frac{1}{2}$ tons upon the rod; fortunately for structures friction between the surfaces in contact comes into play, and if its coefficient be taken at ·2 and introduced into the previous calculations, the $56\frac{1}{2}$ tons will be reduced to the surprisingly small amount of $8\frac{1}{2}$ tons. Nevertheless, in this

way a considerable strain may be put upon short small tension and compression members, especially in structures of moderate dimensions; that is, assuming the portions in compression to be rigid, for if they are not so, no strain whatever can in this way be put upon any portion of the structure, beyond that caused by its own weight and its working load. The term rigid is not here employed in its absolute sense; it is intended to imply the state existing in the compression boom of a bowstring girder. Owing to workmanship not guaranteeing close joints, the elasticity of materials, and other causes, the portions in compression will give to a certain extent; the strain on the iron cannot therefore be deduced from the distance which the nut is screwed up.

Again, the lengths of the various members are often not exactly what they should be; in this case the "drift," although strictly forbidden, is much used, and even heat is sometimes applied to the refractory member.

It is no uncommon thing to hear it stated, that a long bridge will not vary in this country $\frac{1}{32}$nd of an inch in length between the coldest winter and the hottest summer—a conclusion arrived at by putting marks upon a bridge and corresponding ones on the abutments; these, through improper construction, the rusting up of expansion rollers, or other causes, not altering their relative positions are assumed as sufficient proof that the changes of temperature have no effect. If in addition to these marks the camber on a bridge is accurately ascertained, it will be found to vary appreciably owing to changes of temperature if the girders do not move longitudinally. And should the structure be designed and loaded in such a manner that none of these changes can take place, the results will still exist, and under these circumstances may cause the destruction of portions of the structure.

If the difference between the extremes of temperature in this country be assumed at $82 \cdot 5°$ Fahr., and a change

of 15° Fahr. be capable of inducing a strain of one ton per square inch, it follows that this variation in temperature, unless counteracted by some means, will induce a strain of 5·5 tons per square inch. In large girders and similar constructions, protected in many instances by slightly non-conducting paints, the extremes of temperature acting only during a short portion of the day have not time to affect the iron to the extent just stated, an allowance of $\frac{7}{16}$ths of an inch for each 100 feet in length is found sufficient in this country. Now, $\frac{7}{16}$ths of an inch is equal to $\frac{1}{2743}$rd of 100 feet, and as iron within working limits stretches $\frac{1}{10000}$ of length for each ton strain per square inch, it follows that the strain caused in this country in work without proper provision made for changes due to temperature, will be to one ton as $\frac{1}{2743}$ is to $\frac{1}{10000}$, that is, equivalent to a strain of a little over $3\frac{1}{2}$ tons per square inch.

As records from actual practice are always good arguments with which to back up theory, it may be well to quote a few statements. Mr. Clark, in his work on the 'Britannia and Conway Tubular Bridges,' says, when speaking of the former of these bridges:

"Although the tubes offer so effectual a resistance to deflection by heavy weights and gales of wind, they are nevertheless extremely sensitive to changes of temperature, so much so that half an hour's sunshine has a much greater effect than is produced by the heaviest trains or the most violent storm; they are, in fact, in a state of perpetual motion, and after three months' close observation, during which their motions were recorded by self-registering instruments, were never observed to remain at rest for a single hour."

The practical allowance of $\frac{7}{16}$ths of an inch per 100 feet may have been deduced from observations on this bridge. The total length of the Britannia Tube, at 32° Fahr., is

1510′ $1\frac{1}{2}$″, and the variation in length between summer and winter is $6\frac{3}{8}$ inches, or a little less than $\frac{7}{16}$ths of an inch per 100 feet.

Mr. E. A. Cowper, when speaking at the Institution of Civil Engineers, May 5, 1863, made the following remark:

"For instance, Southwark Bridge was erected in cool weather, so that in the full heat of the sun the centre arch rose in the middle $1\frac{3}{4}$ inch; and the various castings forming the spandrils, being deep and well fitted together, without any allowance for expansion, began to break consecutively. The cause of fracture was owing to the stone abutments being fitted tightly against the ends of the spandrils. It had been found necessary to jump holes in the stone, immediately behind the ends of the spandrils, so as to relieve them, and to allow the rib to rise as it expanded. The effects of expansion and contraction in Southwark Bridge were well shown by the hand-railing drawing out of the stone piers in winter, and being thrust into them in summer, the play over the abutments amounting to $\frac{3}{4}$ of an inch. The stone paving was also always out of order over the piers, and in some cases the heavy stones had flushed at their top edges."

The chord of the extrados of this bridge is 240 feet, and the rise 23′ 1″.

In a wrought-iron arch the effect of temperature will be still more appreciable, the coefficient of expansion under strain, when compared with cast iron, being nearly as 1 to 2, and the variations in length due to changes of temperature being not far from equal in both cases; an atmospherical change which will cause a strain of one ton in wrought iron, will therefore induce only half that amount in a cast-iron structure.

This subject might be pursued still further, but sufficient has been given to show that if the iron is prevented from

expanding, the elasticity of the material must take up the would-be expansion; and consequently a greater or less strain put upon the iron, according to whether the change of temperature acts in conjunction or contrary to the original strain in it.

There is no better example to illustrate the effects of the atmosphere than a bridge over a dock passage; here the sun heats the top, whilst the wind, after sweeping along the water, impinges on the bottom flange; which, by keeping it cool, causes it to contract whilst the top is expanding, thus putting no little camber on a girder of the moderate length of 140 feet. To counteract as much as possible this occurrence, which has been frequently witnessed by the author, the top is often painted white and the bottom a darker colour.

Without considering the differences of opinion as to the amount and effect of rolling loads, or the changes in the form of a structure produced by their agency, the remaining circumstances mentioned may be sufficient to often cause the limit of elasticity to be exceeded, although the strain on the iron is supposed to be well within that limit.

Stone is slightly elastic, and affected by changes of temperature, but to so small an extent that in all practical considerations any changes from these causes may be disregarded. With these remarks the author considers that he has conclusively proved that the properties of good ashlar are as well or better known than those of ironwork, and that the strains in stone columns may be determined with more ease and accuracy than in those of iron.

"From the general tenor of the meeting, it appeared that the author's statement concerning thick and thin masonry was not agreed with: it was the opinion that contractors prefer thick whilst engineers lean towards thin walls."

Author's Reply.—The author is still of opinion that, with

proper supervision, thick and thin walls will be of equal quality. In Edinburgh, where most of the houses are of stone, the author has seen walls 2′ 0″ in thickness, to all appearance well built on the faces, constructed in the interior with bad lime, mud, and boulders, and it is seldom that anything less than masonry of this thickness is required in counterforts to retaining walls or other engineering work.

In masonry walls with counterforts, the theoretical economy is seldom obtained, as in order to render the junction of the counterforts with the walls sufficiently strong, fillets must be put in at the angles; the counterforts are therefore generally made much thicker than theory requires and the fillets dispensed with, so that any economical advantages appertaining to their use are not obtained; with brickwork these fillets can be constructed without difficulty and without enhancing the cost of the work, and when dealing with this material counterforts may often be employed with economy.

Although in Scotland thin masonry is the rule, and is advocated by both engineers and architects on account of its superiority, the author, after over a four years' sojourn in that country, has little hesitation in saying that it would be difficult to find north of Berwick many thin rubble walls which will compare with the solid rubble masonry of the Liverpool Docks, often more than 15 feet in thickness. As this good quality is obtained without additional cost, the author is of opinion that efficient specifications and proper supervision by competent men will in the long run obtain the nearest approach to good masonry or ironwork. The principal contractor may make every endeavour to give good work, and supply the best materials obtainable, and yet the work will often be far from what it should; there are generally on large works hosts of sub-contractors, lazy and indifferent workmen, whom to keep up to their work requires the utmost vigilance on the part of both engineer and contractor.

"The style of wall with long counterforts, advocated by the author for use in positions where bad materials require to be supported, or good foundations are not attainable, was stated to have been very successfully employed on railway work in the Highlands."

THE END.

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